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APPLICATIONS FOR ADMISSION AND TRANSFER

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CURRENT PAPERS AND DISCUSSIONS

		Discussion closes
Stresses in Reinforced Concrete Due to Volume Changes. ***C. P. Vetter***	Feb., 1932	
Discussion (Author's closure)	May, Sept., 1932, May, 1933	Closed
The Compensated Arch Dam. ***A. V. Karpov***	Apr., 1932	
Discussion (Author's closure)	Oct., 1932, Mar., May, 1933	Closed
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Discussion (Author's closure)	Nov., 1932, Jan., Feb., May, 1933	Closed
Forests and Stream Flow. ***W. G. Hoyt and H. C. Troxell***	Aug., 1932	
Discussion	Sept., Nov., Dec., 1932, Feb., Mar., Apr., May, 1933	Closed
Tests of Riveted and Welded Steel Columns. ***Willis A. Slater and M. O. Fuller***	Sept., 1932	
Discussion (Author's closure)	Jan., Feb., May, 1933	Closed
The Martinez-Benicia Bridge. ***W. H. Kirkbride***	Sept., 1932	
Discussion	Nov., 1932, Feb., 1933	Closed
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Discussion	Nov., 1932, Jan., 1933	Closed
Tests for Hydraulic-Fill Dams. ***Harry H. Hatch***	Oct. 1932	
Discussion	Jan., Feb., Mar., Apr., May, 1933	Closed
George Washington Bridge: Organization, Construction Procedure, and Contract Provisions. ***Edward W. Stearns***	Oct., 1932	
Discussion (Author's closure)	Dec., 1932, May, 1933	Closed
Foundation Treatment at Rodriguez Dam. ***Charles P. Williams***	Oct., 1932	
Discussion	Mar., 1933	Closed
A History of the Development of Wooden Bridges. ***Robert Fletcher and J. P. Snow***	Nov., 1932	
Discussion	Jan., Feb., Mar., Apr., May, 1933	Closed
Distribution of Shear in Welded Connections. ***Henry W. Troelsch***	Nov. 1932	
Discussion	Feb., Mar., Apr., 1933	Closed
Work of Rivets in Riveted Joints. ***A. Hrennikoff***	Nov., 1932	
Discussion	Mar., Apr., 1933	Closed
Study of Stilling-Basin Design. ***C. Maxwell Stanley***	Nov., 1932	
Discussion	Apr., 1933	Aug., 193
A Problem of Soil in Transportation in the Colorado River. ***S. L. Rothery***	Dec., 1932	
Discussion	Apr., May, 1933	Aug., 193
George Washington Bridge: Design of Superstructure. ***Allston Dana, Aksel Andersen, and George M. Rapp***	Dec., 1932	Closed
George Washington Bridge: Design of the Towers. ***Leon S. Moisseiff***	Dec., 1932	Closed
Model Law for Motion of Salt Water Through Fresh. ***Morrough P. O'Brien and John Cherno***	Dec., 1932	
Discussion	Mar., Apr., May, 1933	Aug., 193
Wind Stress Analysis Simplified. ***L. E. Grinter***	Jan., 1933	
Discussion	Apr., May, 1933	Sept., 193
George Washington Bridge: Construction of Substructure. ***Montgomery B. Case***	Jan., 1933	
Discussion	May, 1933	Closed
George Washington Bridge: Construction of Steel Superstructure. ***E. W. Bowden and H. R. Seely***	Jan., 1933	Closed
Progress Report of the Special Committee on Meteorological Data	Jan., 1933	
Discussion	Apr., May, 1933	Sept., 193
Evaporation from Water Surfaces: A Symposium	Feb., 1933	
Discussion	May, 1933	Sept., 193
George Washington Bridge: Materials and Fabrication of Steel Structure. ***Herbert J. Baker***	Feb., 1933	Closed
George Washington Bridge: Approaches and Highway Construction. ***J. C. Evans***	Feb., 1933	
Discussion	May, 1933	Closed
Developments in Reinforced Brick Masonry. ***James H. Hansen***	Mar., 1933	
Discussion	May, 1933	Sept., 193
The Plan of Boston, Massachusetts: A Capital City. ***Arthur C. Comey***	Mar., 1933	
Discussion	May, 1933	Sept., 193
High Dams on Pervious Glacial Drift. ***Edward M. Burd***	Apr., 1933	
Discussion	May, 1933	Oct., 193
Improved Type of Flow Meter for Hydraulic Turbines. ***Ireal A. Winter***	Apr., 1933	Oct., 193
Three-Span Continuous-Truss Railroad Bridge, Cincinnati, Ohio. ***Wilson T. Ballard***	April., 1933	Oct., 193

CONTENTS FOR MAY, 1933

PAPERS

REPORTS

DISCUSSIONS

CONTENTS FOR MAY, 1933 (Continued)

For Index to all Papers, the discussion of which is current in* PROCEEDINGS, *see page 2

MEMBERSHIP

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

ACTUAL DEFLECTIONS AND TEMPERATURES IN A TRIAL LOAD ARCH DAM

By A. T. Larned[1] and W. S. Merrill[1], Members, Am. Soc. C. E.

Synopsis

This paper contains a description of the layout of the dam of the Ariel Hydro-Electric Development, and the construction methods used in building it; but the following four points are emphasized principally:

1.—The design of the arch dam, and the effect of the cement content and the manner and speed of construction on its thickness and the resulting stresses;

2.—The proportioning of the concrete to secure good strength with a minimum of setting-heat generation;

3.—The amount of heat generated in the concrete mass during setting and how it was dissipated with time; and,

4.—The radial and tangential deflections of the arch.

It is shown that the amount of heat generated in a concrete mass can be calculated closely so that the construction methods and schedule can be arranged accordingly. If necessary the cooling can be accelerated without harm to the concrete. It is also shown that the concrete in the interior of a mass, where the setting heat is greatest, is as strong as, or stronger than, that nearer the surface. There is a satisfactory agreement between the actual and the calculated radial and tangential deflections.

Introduction

The Ariel Hydro-Electric Development is on the north fork of the Lewis River, at Ariel, Wash., about 25 miles north of Portland, Ore. The location is clearly indicated in Fig. 1. The initial installation consists of one 45 000-kw unit under a static head, with the reservoir full, of 185 ft. The

Note.—Discussion on this paper will be closed in September, 1933, *Proceedings*.

[1] Hydr. Engrs., Electric Bond and Share Co., New York, N. Y.

power house is situated in and over the old stream bed immediately below the arch portion of the dam. The arch dam has a maximum height of 313 ft and contains approximately 200 000 cu yd of concrete. A general view of the dam and power house is shown by Fig. 2.

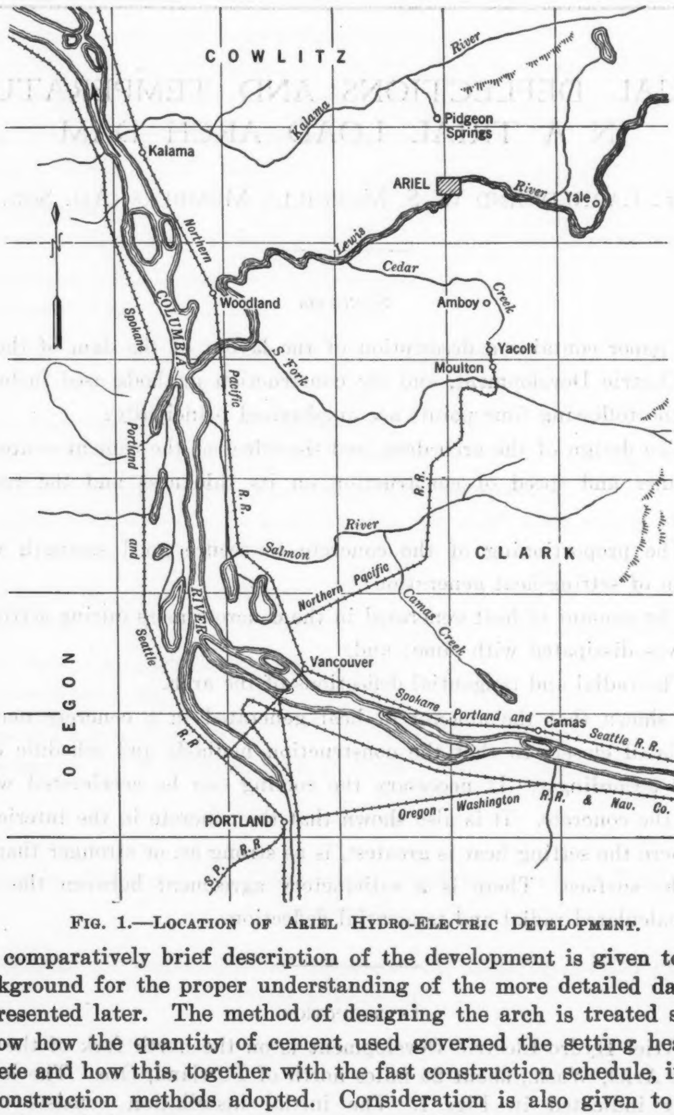


FIG. 1.—LOCATION OF ARIEL HYDRO-ELECTRIC DEVELOPMENT.

A comparatively brief description of the development is given to provide a background for the proper understanding of the more detailed data which are presented later. The method of designing the arch is treated somewhat to show how the quantity of cement used governed the setting heat of the concrete and how this, together with the fast construction schedule, influenced the construction methods adopted. Consideration is also given to the proportioning of the concrete and to the special tests made to determine the strength of concrete taken from the dam itself by means of 4-in. diamond-drill cores.

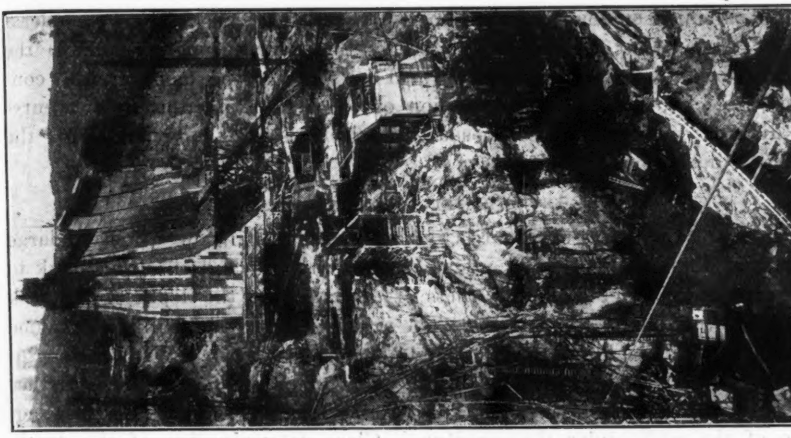


FIG. 3.—EXCAVATION OF DEEP CUT IN RIVER.

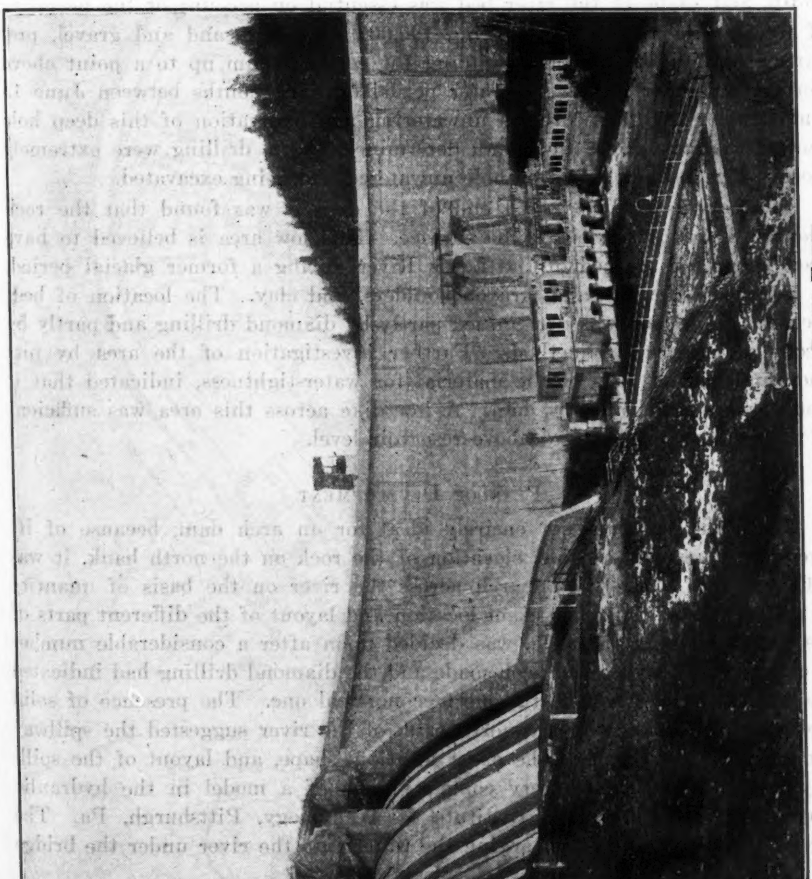


FIG. 2.—GENERAL VIEW OF DAM AND POWER HOUSE.

The principal emphasis of the paper, however, is given to (1) a consideration of the amount of heat generated by the setting concrete and to the methods used to help dissipate this heat in some of the more rapidly constructed blocks; and (2) the deflection of the arch as measured at twenty-one different points. These measured deflections are compared with the computed deflections.

Investigations and Studies

Previous to starting the final layout of the dam and power house a large amount of diamond drilling was done in the river bed and on either bank to determine the character and tightness of the rock and the depth of the rock below the water surface in the river. The greater part of the drilling was done on lines at right angles to the river and about 100 ft apart. With the beginning of the detailed location and design of the dam and power house further drilling was done in the river bed and on either bank to determine the exact shape of the rock under the structure. Close determination of the depth, width, and shape of the river bed was essential on account of the necessity of unwatering the river, removing 190 000 cu yd of sand and gravel, preparing the foundations, and building the concrete dam up to a point above normal tail-water in the low-water period of four months between June 15 and October 15, 1930. Actual unwatering and excavation of this deep hole showed that the shape and depth determined by the drilling were extremely close. Fig. 3 shows this deep hole unwatered and being excavated.

Beyond the extreme north end of the dam it was found that the rock dropped about 100 ft below the surface. This low area is believed to have been a part of the bed of the Lewis River during a former glacial period. It is now filled with sand, gravel, boulders, and clay. The location of bed-rock in this section was determined partly by diamond drilling and partly by electrical prospecting methods. Further investigation of the area by pits and tunnels, and tests of the material for water-tightness, indicated that it would make a satisfactory dam. A low dike across this area was sufficient to provide ample free-board above reservoir level.

Plan of Development

While the site was not entirely ideal for an arch dam, because of its irregular shape and the low elevation of the rock on the north bank, it was sufficiently so to justify an arch across the river on the basis of quantity of material and cost. The exact location and layout of the different parts of the dam, including the arch, was decided upon after a considerable number of comparative studies had been made and the diamond drilling had indicated the position selected to be the most economical one. The presence of solid rock near the surface on the north bank of the river suggested the spillway layout shown by Fig. 4. The exact location, shape, and layout of the spillway were determined by very complete tests of a model in the hydraulic laboratory of the Carnegie Institute of Technology, Pittsburgh, Pa. The layout adopted permits discharging the water into the river under the bridge

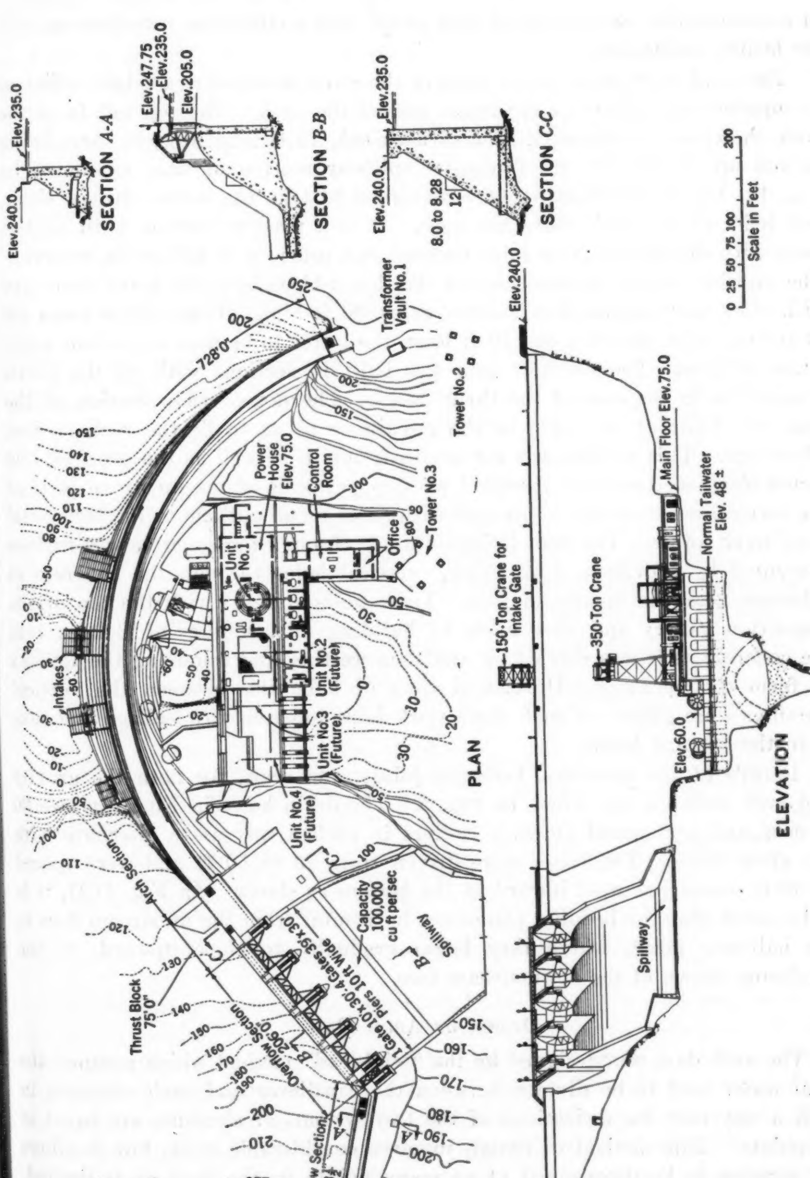


FIG. 4.—PLAN AND ELEVATION OF DAM AND POWER HOUSE.

in a manner that allows a good "get-away" and a minimum interference with the bridge abutments.

The solid rock on the north bank of the river extended to a height sufficient to support only a part of the north end of the arch. The top 140 ft of the arch, therefore, is supported by a thrust-block, 75 ft long, this, in turn, being backed up by the 211 ft of massive spillway section of dam as shown by Fig. 4. The thrust-block itself is designed to take the entire thrust, shear, and twist of the arch above the rock. It is a massive section built in two parts with the construction joint grouted just previous to filling the reservoir. The overflow section of dam beyond the thrust-block is of the usual ogee type with piers that support five Taintor gates, 30 ft high. Four of the gates are 39 ft long, with the fifth one 10 ft long, the latter to be used for minor regulation of flow. The overflow dam was built in sections with all the joints grouted, as in the case of the thrust-block. The non-overflow section of the dam was built at an angle to the remainder so as to fit the rock to best advantage. This section was not grouted, but it, as well as the overflow and thrust-block sections, was provided with copper seals at the up-stream side of the vertical construction joints and with sheet metal cut-offs at the horizontal days' work joints. The percolation of water through the concrete was further prevented by providing 4-in. square, vertical holes through the concrete at intervals near the up-stream face. These holes connect with the top of the inspection gallery and were made by building 4-in. squares of Celotex into the concrete as it was carried up and then wetting and removing the Celotex to form the openings. Horizontal rows of half tiles between the vertical openings were placed on each day's work joint to catch any leakage and take it to the vertical holes.

Details of the grouting, keyways, joints, and seals, for both the gravity and arch sections, are shown by Fig. 5. The drain holes in Fig. 5(*a*) are 30 ft deep and are spaced on 20-ft centers in such a way as to alternate with the grout holes. The latter were driven from 25 to 50 ft and were spaced on 10-ft centers, slanted inward at the bottom as shown. In Fig. 5(*d*), it is to be noted that the bedding planes are horizontal from the up-stream face to the half-way point, where they begin gradually to slope upward, to the maximum shown at the down-stream face.

Design of Arch Dam

The arch dam was designed by the "trial-load" method which assumes the total water load to be divided between the cantilever and arch elements in such a way that the deflections of the two systems of elements are equal at all points. This method of design involves considerable work, but it allows the stresses to be determined at as many points in the dam as is desired, this number being limited by the time and money allowed. This method has been used by the United States Bureau of Reclamation in designing and checking several important arched gravity dams. It permits making allowance for foundation and abutment deflections, for radial sides of cantilever elements, for loads carried by tangential shear and twist, seasonal variation in

concrete temperatures, up-stream deflection of the dam when the reservoir is empty, and other miscellaneous factors, as well as moment, transverse shear, and thrust in both the arch and cantilever elements, which are usually considered. No allowance was made for the effect of water-soaking of the concrete at the up-stream face of the dam, for Poisson's ratio, or for probable reduction in maximum stresses due to the flow of concrete.

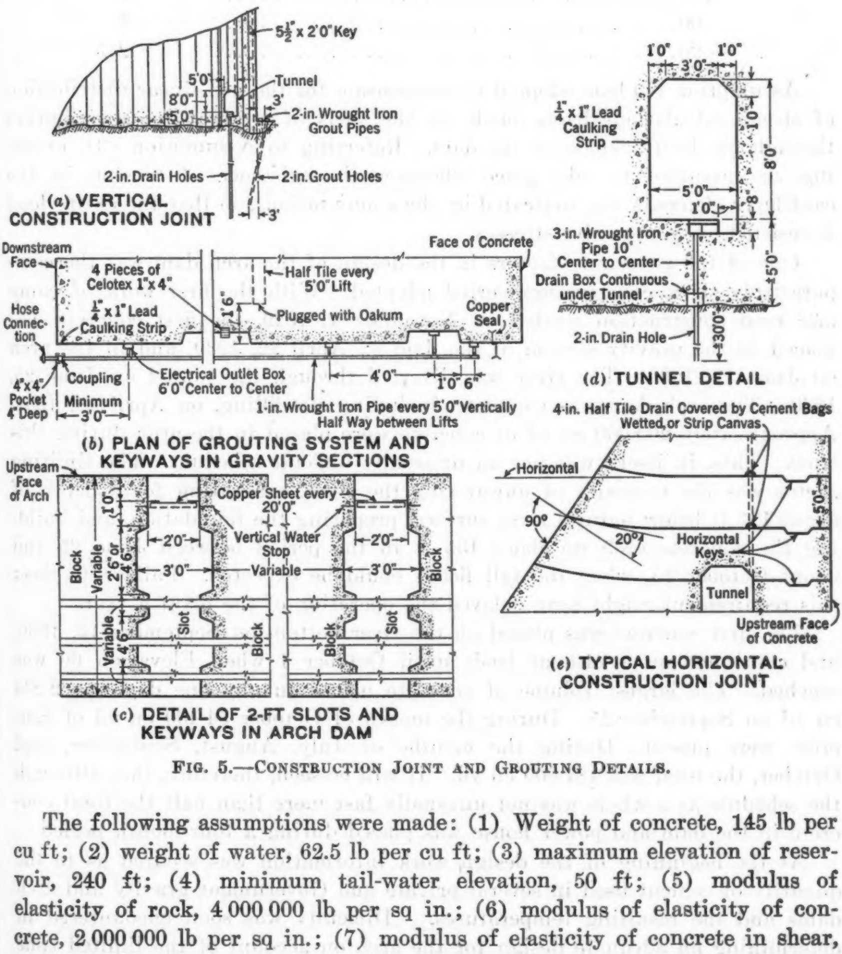


FIG. 5.—CONSTRUCTION JOINT AND GROUTING DETAILS.

The following assumptions were made: (1) Weight of concrete, 145 lb per cu ft; (2) weight of water, 62.5 lb per cu ft; (3) maximum elevation of reservoir, 240 ft; (4) minimum tail-water elevation, 50 ft; (5) modulus of elasticity of rock, 4 000 000 lb per sq in.; (6) modulus of elasticity of concrete, 2 000 000 lb per sq in.; (7) modulus of elasticity of concrete in shear, 96 000 000 lb per sq in.; (8) maximum allowable vertical or horizontal compressive stress of concrete, 600 lb per sq in.; (9) maximum allowable tensile stress of concrete in arch elements, 100 lb per sq in.; (10) the uplift pressure at the base of the arch section varies from full reservoir pressure at the up-stream side to zero or tail-water pressure, at the down-stream side (these pressures to be applied to one-half the horizontal area of the base); (11)

cement content, 1 bbl per cu yd; (12) temperature drop in arch concrete after the slots have been closed must not exceed the following:

Elevation	Temperature drop, in degrees Fahrenheit
220	11
180	9
120	5
60	3
− 20	2.5

Assumption (7) was adopted to compensate for the non-linear distribution of shear, calculations being made on the basis of straight-line distribution throughout the thickness of the dam. Referring to Assumption (9), cracking was assumed to take place wherever the existence of tension in the cantilever elements was indicated by the computations, so that the entire load is resisted by compressive stresses.

One of the governing factors in the design of the arch dam was the comparatively short construction period adopted. With the first work of camp and road construction started on November 1, 1929, the first concrete was placed in the gravity section of the dam on April 22, 1930, and in the arch on June 11, 1930. The river was diverted through the tunnel on June 25, 1930. The arch dam was complete, including slot-filling, on April 18, 1931. Approximately 200 000 cu yd of concrete were placed in the arch during this time. This, in itself, was not an unusually difficult schedule. The limiting factor was the necessity of unwatering the river, excavating for a depth of about 125 ft below natural river surface, preparing the foundation, and building the concrete arch up about 135 ft in the period between June 25 and about October 15, when the fall floods could be expected. Failure to meet this requirement might have delayed the operation of the plant a year.

The first concrete was placed on the river bottom on September 13, 1930, and continued on a 24-hour basis until October 1 when Elevation 60 was reached. The largest volume of concrete placed in any one day was 2 234 cu yd on September 25. During the month of October 54 460 cu yd of concrete were placed. During the months of July, August, September, and October, the total was 181 000 cu yd. It will be seen, therefore, that although the schedule as a whole was not unusually fast more than half the total concrete in the dam and power house was placed during a four-month period.

At the beginning of the design work information was secured as to the quantity of cement used in several private and Government gravity and arch dams and the resulting temperatures. Difficulty was soon encountered in determining an adequate design for the arch on account of the limited time available to dissipate the setting heat of the concrete. It was found that thickening the cross-section to reduce the load stresses increased: (1) The setting heat; (2) the temperature drop to be taken care of in the design; and (3), the temperature stresses produced in the arch. The increases in tensile stress produced at the extrados at the abutments and at the intrados at the crown, due to the increased temperature drop, were especially undesir-

able. Finally, it was found necessary to adopt the block construction instead of the grouted-joint construction in order to dissipate the setting heat in the limited time available. The allowance for the temperature drop with this method of construction is indicated in the tabulation under Assumption (12). Division into comparatively short blocks, with 2-ft slots between (see Figs. 2 and 3), permitted the development of an economical and rational design which will not be subject to unreasonably high secondary stresses.

The arch is divided into blocks approximately 30 ft long (see Fig. 6), except in that portion behind the power house where the penstocks come through, where the blocks have a length of about 40 ft at the up-stream face of the dam. The openings, or slots, between blocks, were kept as short as possible for two reasons: First, to keep to a minimum the quantity of concrete that would have to be placed rapidly at the very end of the job; and, second, to minimize the heat generation that would come from this volume of new concrete. The 2-ft slots adopted were just sufficient to allow proper handling of the end forms and also to allow for the proper placing of the final slot concrete.

The stresses in the arch rings were computed at forty-five different points at five different elevations. At each elevation the stresses were determined at the crown and abutments and at three points between. The stresses finally computed in the arch rings, adjusted for the effects of tangential shear and twist, are shown by Table 1. The stresses finally computed in the cantilever elements, adjusted for the effects of tangential shear and twist, are shown by Table 2. The stresses are shown for both the up-stream and down-stream faces of the dam in both the vertical and inclined directions. In several cases the cantilever section is indicated as having a crack extending a considerable distance in from the up-stream face. In those cases the compression is computed as zero at the end of the crack, the total compression being taken by the remainder of the section. It will be seen from Tables 1 and 2 that the maximum stress in the arches is considerably less than 600 lb per sq in. in every case, with one point showing a maximum of 560 lb per sq in. The maximum stresses in the dam occur in the cantilever elements, the 600-lb limit being exceeded slightly in two instances.

Construction Details of Arch Dam

The arch dam extends between the rock abutments on either side of the river with the up-stream radius varying from 397 ft at the top to 247 ft at Elevation 40. The angle between the ends of the arches varies between 108° at the top and 80° at normal tail-water at Elevation 50. The plan and maximum section of the arch are shown in Figs. 6 and 7. It will be noted that the maximum length of the arch at the top is 728 ft, the maximum height, 313.4 ft, and the maximum thickness, 93 ft. Although the arch is vertical at the up-stream side at its deepest part it does overhang about 5 ft near the abutments. Fig. 6(*a*) also shows the four main penstocks, the house unit penstock, and the location of the various slots. The lines shown for the slots indicate their intersection with the outside faces of the concrete. In every case the slots extended radially through the dam at each 10-ft elevation.

TABLE 1.—STRESSES IN ARCH ELEMENTS, IN POUNDS PER SQUARE INCH
(Reservoir elevation, 240; + = compression; — = tension)

Elevation	North Abutment		Three-Fourth Point		Half-Point		Quarter-Point		Crown		Quarter-Point		Half-Point		Three-Fourth Point		South Abutment	
	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos
220....	+166	+233	+314	+87	+87	+294	—28	+417	+81	+312	+255	+140	+349	+48	+267	+129	+171	+220
180....	+560	—107	+325	+122	+49	+392	—21	+46	+145	+298	+286	+160	+348	+99	+294	+153	+72	+370
120....	+372	—105	+182	+76	+33	+221	—33	+284	+31	+223	+142	+116	+216	+44	+201	+59	+64	+192
60....	+402	—16	+408	—14	+299	+131	+117	+335	+21	+441	+117	+335	+299	+131	+408	—14	+402	—16
—20....	+402	—43	+292	+77	+185	+203	+95	+323	+61	+369	+95	+323	+185	+203	+292	+77	+402	—43

TABLE 2.—STRESSES IN CANTILEVER ELEMENT, IN POUNDS PER SQUARE INCH (SEE FIG. 10)*

Cantilevers	*A*		*C*		*E*		*G*		*I*		*K*		*M*	
Direction of stress	Inclined	Vertical	Inclined	Vertical	Inclined	Vertical	Inclined	Vertical	Inclined	Vertical	Inclined	Vertical	Inclined	Vertical
							UP-STREAM FACE							
Elevations:														
220..................	+36	+36	+41	+41	+42	+42	(9)	(9)	+37	+37	+34	+34	+39	+39
180..................	+73	+73	+95	+94	+111	+110	+94	+93	+75	+74	+56	+56	+80	+80
140..................	+10	+10	+49	+38	+68	+66	+21	+20	+ 5	5	(9)	(9)	+20	+20
120..................	(13)	(13)	(4)	(4)	+25	+25	(105)	(105)	(16)	(16)	(21)	(21)	(11)	(11)
110..................	....	....	....	....	....	....	....	....	....	....	(33)	(33)	(30)	(30)
100..................	(17)	(17)	(16)	(16)	(11)	(11)	(23)	(23)	(22)	(22)	....	....	....	....
90..................	(34)	(34)	....	....	....	....	....	....	....	....	....	....	....	....
75..................	....	....	....	....	....	....	....	....	(45)	(45)	....	....	....	....
60..................	....	....	(27)	(27)	(28)	(28)	(23.5)	(23.5)	....	....	....	....	....	....
20..................	....	....	(35)	(35)	(33)	(33)	(36)	(36)	....	....	....	....	....	....
0..................	....	....	(56)	(56)	....	....	(55.5)	(55.5)	....	....	....	....	....	....
—20..................	....	....	....	....	(31)	(31)	....	....	....	....	....	....	....	....
—50..................	....	....	....	....	(42)	(42)	....	....	....	....	....	....	....	....
							DOWN-STREAM FACE							
220..................	+37	+37	+41	+41	+43	+43	+41	+41	+37	+37	+33	+33	+39	+39
180..................	+71	+71	+95	+94	+113	+112	+99	+98	+74	+73	+55	+55	+80	+80
140..................	+151	+144	+111	+108	+94	+93	+147	+145	+162	+159	+177	+172	+136	+127
120..................	+250	+238	+197	+193	+174	+172	+250	+247	+295	+292	+333	+323	+230	+211
110..................	....	....	....	....	....	....	....	....	....	....	....	....	+451	....
100..................	+298	+248	+303	+291	+272	+269	+414	+409	+396	+341	+540	+337	....	+414
90..................	+440	....	....	....	....	....	....	....	....	....	....	....	....	....
75..................	....	....	....	....	....	....	....	....	+808	....	....	....	....	....
60..................	....	+324	+441	+324	+438	+421	+511	+297	....	+647	....	....	....	....
20..................	....	....	+450	+432	+533	+512	+470	+452	....	....	....	....	....	....
0..................	....	....	+647	....	....	....	....	....	....	....	....	....	....	....
—20..................	....	....	....	+622	+571	+549	+632	+607	....	....	....	....	....	....
—50..................	....	....	....	....	+584	+561	....	....	....	....	....	....	....	....

* Numerals in parenthesis denote points at which crack occurred and at which stress is zero; thus, (13) denotes a crack extending 13 ft from the face.

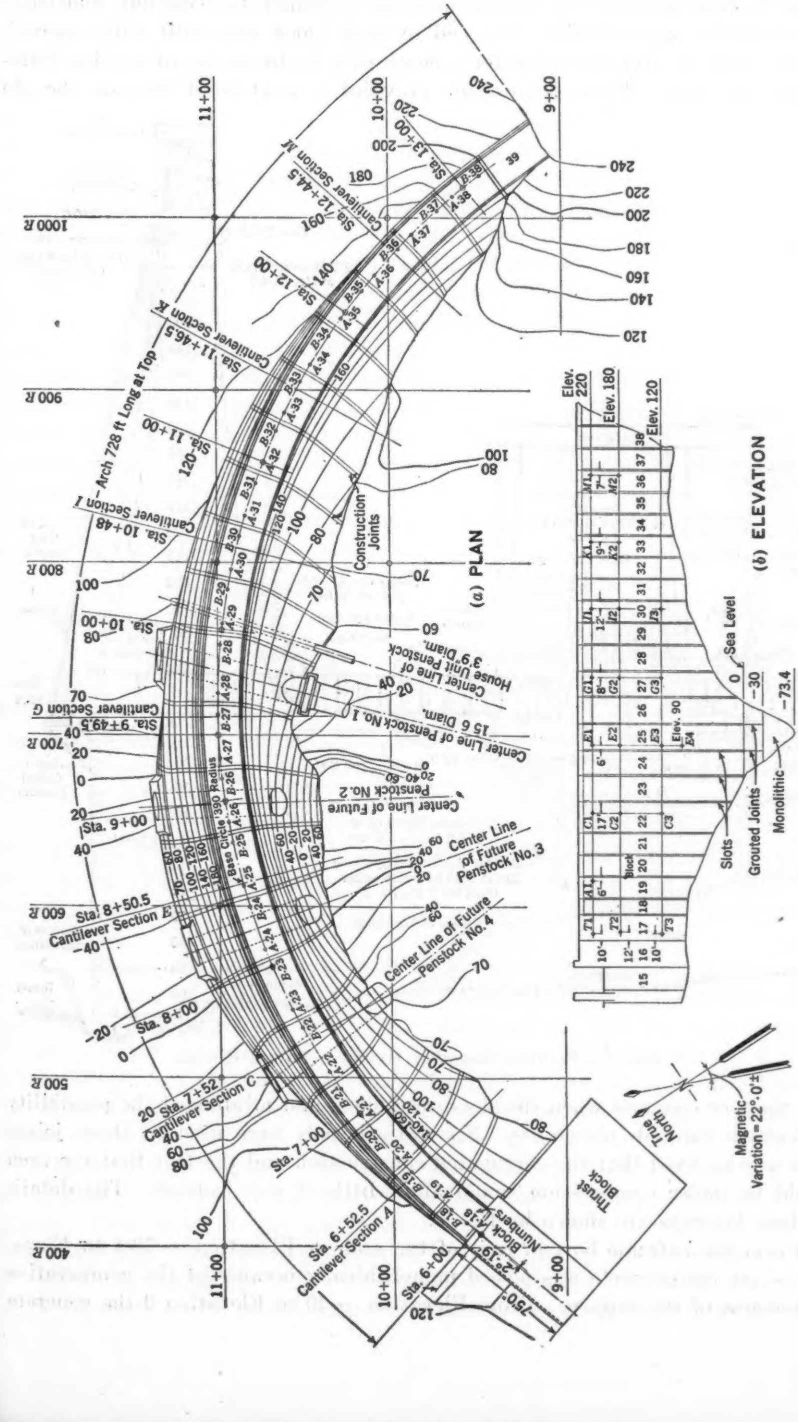


FIG. 6.—BLOCK NUMBERING AND LOCATION OF DEFLECTION.

This resulted in curiously warped openings in which the concrete sometimes overhung for several feet. The end of each block was built with approximately half its area depressed for a depth of 6 in. by means of wooden forms 4 ft 6 in. long. These depressions provided a good bond between the old

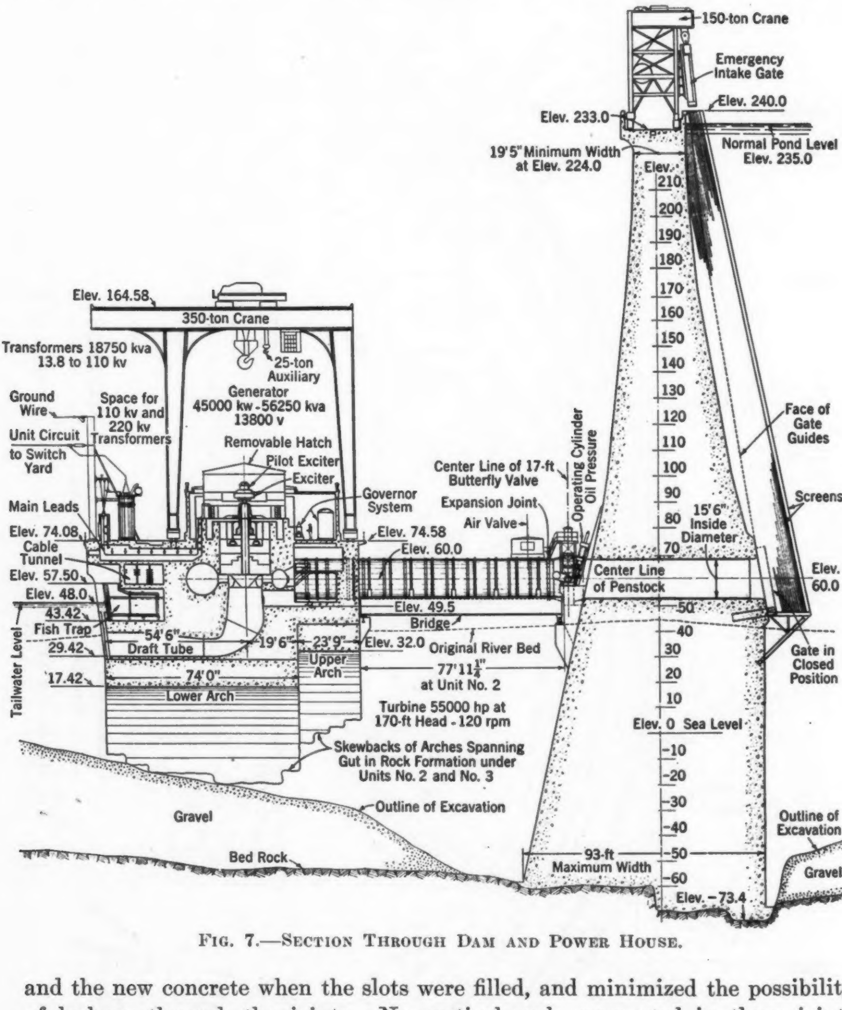


FIG. 7.—SECTION THROUGH DAM AND POWER HOUSE.

and the new concrete when the slots were filled, and minimized the possibility of leakage through the joints. No vertical seals were used in these joints as it was believed that the corrugated construction and the fact that the arch would be under compression would allow little if any leakage. The details of these keyways are shown by Fig. 5.

From the extreme bottom part of the arch at Elevation − 73.4 to Elevation − 30, the concrete was placed monolithically because of the comparative narrowness of the canyon. From Elevation − 30 to Elevation 0 the concrete

was built in four blocks with vertical construction joints between. These joints were provided with a piping system for grouting, the piping being placed horizontally at each 5-ft elevation and opening into electrical conduit boxes placed against the old concrete at intervals of 4 ft 6 in. on the face of each block. Above Elevation 0 the arch was built in blocks with the 2-ft slots between, except for Blocks 20, 21, and 22 (Fig. 6), which were left low through the winter of 1930-31 for the purpose of passing excess flood-waters that could not be handled by the diversion tunnel. These blocks were finally built with only construction joints between them. These joints also were provided with a piping system for grouting after they had been cooled.

The foundation of the arch was thoroughly grouted at the up-stream face by means of diamond drill holes extending 20 to 50 ft into the rock on 10-ft centers. This grouting, as well as the grouting of the joints, noted previously, was done just before filling the reservoir, on May 13, 1931. The top of the arch on the down-stream side has a simple curved cornice for architectural effect. In the top 7 ft considerable concrete was saved by building only comparatively thin walls at the up-stream and down-stream sides. The wall at the up-stream side is provided with copper seals at each joint, while the down-stream wall has asphalt strips in the construction joints to allow for possible expansion and contraction.

Construction Methods and Materials

For the purpose of building the arch dam and power house, the river bed was unwatered by means of rock-filled timber cribs built on the river gravel above and below these structures. The total volume of material removed was 190 000 cu yd, 40 000 cu yd of this being taken out before the river was unwatered, by means of derricks on the river banks. Some of the gravel removed from the river bed at the dam was used for concrete aggregate. The main gravel deposit in the bar below the lower coffer-dam was removed from the river by means of buckets on two slack-line cableways covering the entire area involved. These cableways dumped into a hopper on the north river bank. The gravel was carried by a belt conveyor to the screening and washing plant where it was washed and graded into three sizes and stored in piles.

All cement was delivered in bulk to the contractor's siding twelve miles from the plant. It was hauled to the plant by trucks having specially constructed 50-bbl bodies provided with compressed-air jets for loosening the cement during unloading. It was found, however, that the use of air was unnecessary. At the job these trucks entered a cement shed and dumped the cement directly into storage bins. From the storage bins, the cement was taken to the top of the concrete mixing plant by a belt conveyor.

The sand and the two sizes of concrete aggregate were taken from the stock piles to the top of the concrete mixing plant separately by a belt conveyor. Each size of aggregate and the cement were accurately weighed separately and then discharged into 2-yd mixers for $2\frac{1}{2}$ min of mixing. The concrete was discharged into 2-yd bottom-dump buckets on small industrial cars. The cars were hauled from the mixer to the placing point at the dam

by gasoline locomotives. The buckets were handled by derricks to some extent, but mostly by "whirleys." Each block of the arch was built up with the whirley located on an adjacent block. The work was carried on from each end and at two levels, the whirleys moving on to the newly completed block about two days after its completion. The whirleys and derricks spotted the concrete buckets accurately over the place where the concrete was finally to rest. Almost no rehandling of the concrete was necessary or allowed.

The arch blocks were built with unlined wooden forms, 10 ft. high by 12 ft long. This length was short enough so that the chords closely approached the circular arcs on which the dam was laid out. The forms were placed in proper position by measurements from a base circle on which there were two transit points in each block. Six angle and distance measurements from each of the two points were sufficient to determine the location of the forms accurately. The necessary angles and distances for each 10-ft elevation were prepared in tabular form, the distances being scaled from a large scale plan of the dam and the angles computed therefrom. Experience in the design of a previous arch dam in which all distances and angles were calculated and then checked by measurement from the drawing indicated that the scaling of the distances from the drawing and the calculation of the angles was sufficiently accurate.

Concrete Proportioning and Strength

The sand and coarse aggregate were secured partly from the excavation in the river bed at the dam, but mostly from a gravel bar about $\frac{1}{4}$ mile down stream. The aggregate consisted of well rounded particles of andesite and basalt lava, most of the rock being dense and heavy, but some of it being somewhat vesicular. The vesicular rock and pumice were largely removed in the washing and screening process. The specific gravity of the sand averaged 2.47 and that of the coarse aggregate, 2.38. The aggregate as a whole had an absorption value of 3.9% in 24 hr. With the exception of being slightly light in weight the aggregate was sound and entirely suitable for concrete. The weight of saturated concrete in place in the dam, as determined from a considerable number of samples removed after some months, was about 147.5 lb per cu ft

The concrete in the dam had a nominal mix of 1:$2\frac{1}{2}$:5. To attain maximum density, it was proportioned by weight, as follows:

Mix by volume	Material weights for 2-yd batch
1 part cement	Cement, 752 lb
2.4 parts sand, 0 in. to $\frac{5}{16}$ in.	Sand, 1 900 lb
2.0 parts pebbles, $\frac{5}{16}$ in. to $1\frac{1}{2}$ in.	Pebbles, 1 450 lb
4.6 parts cobbles, $1\frac{1}{2}$ in. to 4 in.	Cobbles, 3 450 lb

Water, 64 gal.

The materials used for making the concrete, and the concrete itself, were continuously inspected by a special force of four men. The inspectors had complete control of the concrete mixture and its handling and placing in the dam. In addition, they were responsible for the proper preparation of the

concrete surfaces on which new concrete was to be placed and for the proper handling of the seals, cut-offs, drains, etc. The testing of the cement at the mills and at the laboratory, and the testing of the concrete materials and the concrete itself were done in accordance with A. S. T. M. standard methods in a modest but well equipped testing laboratory at the dam. A total of 379 831 bbl of cement were delivered to the job in bulk and its acceptable quality was determined by a total of 1 969 tests. The physical characteristics of the aggregate and the concrete made from it are given in Table 3. These data are typical of those attained throughout the job.

TABLE 3.—Typical Test Report on Concrete and Aggregate

Date September 30, 1930

Serial No. 127 Location Block 24—Elev. 40–50.

Temperature Max 74° F. Min 41° F. Test represents 1506 Cu. Yds. Concrete

SIEVE ANALYSIS
AGGREGATE

Sand				Fine Gravel				Coarse Gravel			
Sieve No.	Weight	Percent. Retained	Percent. Greater	Sieve No.	Weight	Percent. Retained	Percent. Greater	Sieve No.	Weight	Percent. Retained	Percent. Greater
4	....	.....	.....	3"	....	.....	.....	3"	....	.....	.....
8	50	10.0	10.0	2"	....	.....	.....	2"	....	.....	.....
14	105	21.0	31.0	1½"	....	.....	.....	1½"	23	70	70
28	179	35.8	66.8	1"	....	.....	.....	1"	....	.....	.....
48	118	23.6	90.4	¾"	12	35	35	¾"	9	27	97
100	39	7.8	98.2	⅜"	13	38	73	⅜"	1	3	100
—100	9	1.8	100.0	No. 4	9	27	100	No. 4	....	.....	.....
Total	500	100.0	296.4	.....	34	100	208	.....	33	100	267
Fineness Modulus....			2.96				7.08				867
Percent. of Mix......			27				22				51

Mixed Aggregate

Sieve No.	Sand	Fine Gravel	Coarse Gravel	Final
100.......	98	100	100	99
48.......	90	100	100	97
28.......	67	100	100	91
14.......	31	100	100	82
8.......	10	100	100	76
4.......	..	100	100	73
⅜".......	..	73	100	67
¾".......	..	35	97	58
1½".....	..	...	70	36
3".......	..	...	...	..
F. M.....	2.96	7.08	8.67	6.78

Sand Test

	Compressive	Tensile
Local....	lbs.	lbs.
7 day....	....	....
28 day...	....	....
Average..	....	....
Standard.	....	....
7 day....	....	....
28 day...	....	....
Average..	....	....

Miscellaneous Properties

Aggregate

	Sand	Fine Gravel	Coarse Gravel	Final
Voids..........	35.0	36.2	34.0	18.0
Wt lbs cu ft......	104	96	98	124
Spec. Gr.......	2.57	2.41	2.38	2.43
% Abs. 24 Hrs..	2.0	4.6	4.4	3.8
% Moisture....	5.9	4.1	1.3	3.2
% Loss by Dec..	1.4	.....	.....	......
Soundness......	O.K.	O.K.	O.K.	......
Colm. No......	1	.....	.....	......

Concrete

Mix.................	1–2.4–2.0–4.6
Bbls Per Yard..........	1.0
w/c..................	1.05
Density..............	86.8
Relative Consistency....	1.1
Slump..................	1"

Test

Comp. Str......	7 da	28 da	60 da
Lbs. per sq inch..	1 660	2 810	3 180
Comp. Str.......	3 mo	6 mo	....
Lbs. per sq inch .	3 530	3 750	....

Remarks:

Two hundred and fifty-three 6 by 12-in. test cylinders were made during the construction of the dam and tested for compressive strength, in the laboratory, at various ages. The average strength of the various cylinders at 7, 28, 60, 90, and 180 days is shown by Fig. 8, from which it will be seen that the average compressive strength of the concrete was about 2 800 lb per sq in., at 28 days, and 3 750 lb per sq in., at 180 days.

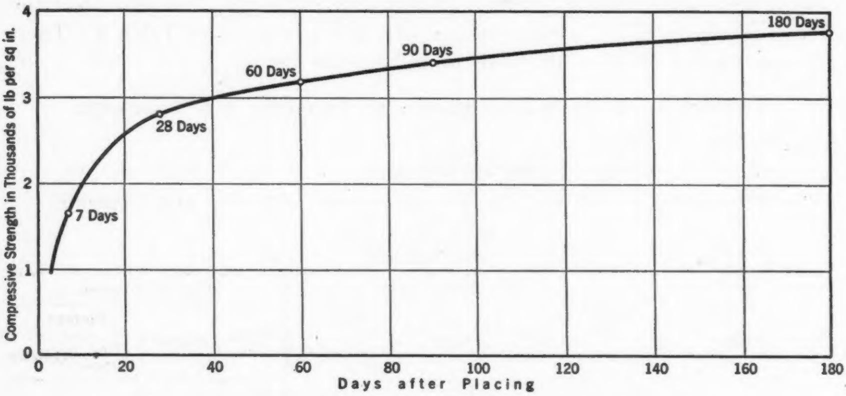


FIG. 8.—TEST CYLINDERS—AVERAGE COMPRESSIVE STRENGTH.

4-INCH DIAMOND DRILL CORES FROM INTERIOR OF DAM

For a long time there has been a question as to whether the results obtained from laboratory tests of concrete cylinders are representative of the strength of the concrete in place in a structure. To determine this point for the Ariel Dam six holes were drilled into it at different locations, angles, and depth by means of a diamond drill using a special barrel which recovered $3\frac{1}{16}$-in. cores, 5 ft long. A total drilling of 253 ft gave 243 ft 8 in. of core available for testing. This core was cut into 258 cylinders 8 in. long and the remainder was used for determining moisture content and weight. The location in elevation of one of these six holes is shown by Fig. 9. Some

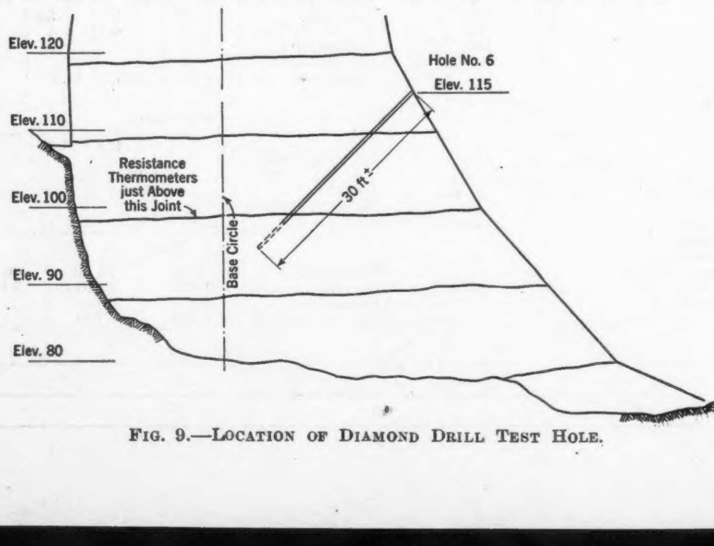


FIG. 9.—LOCATION OF DIAMOND DRILL TEST HOLE.

of these holes were drilled at an angle with the horizontal so as to cut across the days' work joints to determine the bonding of the old and new concrete at these points.

The high percentage of core recovery indicates the excellent quality of the concrete; many cores were obtained the full length of the core barrel (5 ft), and considerable difficulty was encountered at times in breaking off the core. In all the holes only one batch of honey-combed concrete, 8 in. thick, was encountered. An excellent bond existed between the various days' work joints. These joints could be identified in only a few places and then only when different brands of cement were used. Fig. 10 shows the weight and

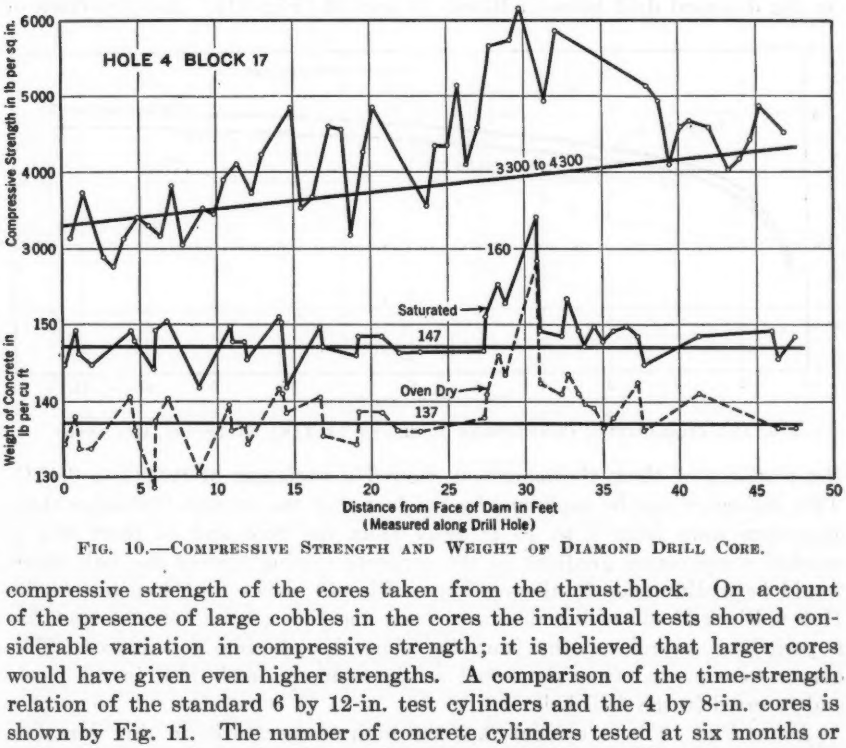


FIG. 10.—COMPRESSIVE STRENGTH AND WEIGHT OF DIAMOND DRILL CORE.

compressive strength of the cores taken from the thrust-block. On account of the presence of large cobbles in the cores the individual tests showed considerable variation in compressive strength; it is believed that larger cores would have given even higher strengths. A comparison of the time-strength relation of the standard 6 by 12-in. test cylinders and the 4 by 8-in. cores is shown by Fig. 11. The number of concrete cylinders tested at six months or earlier is shown, for each point on the curves, in Table 4. The core specimens

TABLE 4.—NUMBER OF CONCRETE CYLINDERS AVERAGED AT VARIOUS AGES (SEE FIG. 10)

Point No.	Number of Cylinders Averaged	
	Cement No. 1	Cement No. 2
1	60	193
2	60	193
3	60	193
4	14	8
5	14	3

were tested at ages greater than nine months. In Fig. 10 the number beside the triangle shows the core hole (Fig. 8) from which the point is taken. While the results indicate that the average strength of some of the cores is less than might be expected from the curve exterpolated beyond 180 days it is believed that these irregularities are explainable by the presence of the large cobbles.

The resistance thermometers embedded in the concrete, close to these holes, showed temperatures in the concrete around the holes as great as 61.9° C (144° F), and a temperature of 50° C (122° F) was common. To test the accuracy of the resistance thermometers, mercury thermometers were placed in the diamond drill holes in Blocks 17 and 32 (Fig. 11). A comparison of

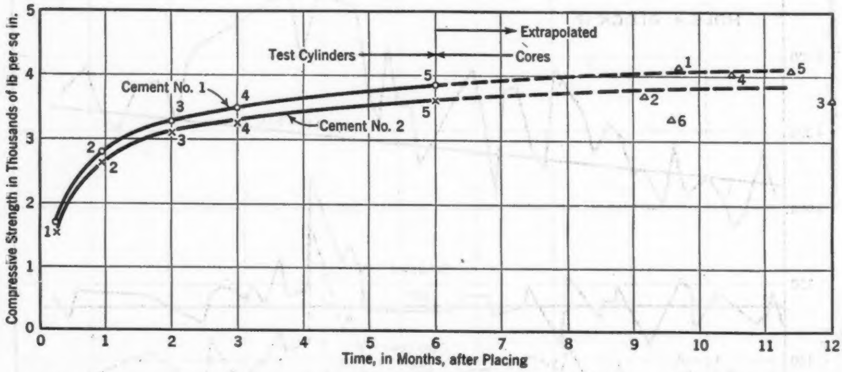


FIG. 11.—COMPARATIVE COMPRESSIVE STRENGTH OF TEST CYLINDERS AND CORES.

the readings of these thermometers showed a variation of 0.9° C to 3.0° C. This difference can be explained by the fact that the various resistance thermometers were from 4 to 15 ft away from the hole and as there was a marked temperature gradient in the concrete sloping toward the hole there would be a difference in the readings. Fig. 12 shows the temperatures in Hole 6, Block 32, at varying distances from the face of the concrete and for a period of several months from the time the concrete was placed. The curves were plotted from data read on thermometers embedded in the block and inserted in the drill hole.

From Fig. 10 it is evident that the high temperatures in this dam (as great as 61.9° C) have caused no loss in the strength of the concrete subject to that temperature in the interior of the dam. On the contrary, it is evident that concrete 9 to 12 months old increased in strength from the surface inward and also from the foundations upward. At and near the foundations lower temperatures prevailed in the concrete due to the cooling effect of the rock. These results seem to be well corroborated not only by Fig. 10, but by the results of the tests of the cores from all the holes.

In the process of obtaining the diamond drill cores they became so thoroughly wet that the moisture content of the concrete in place in the dam could not be determined from these core specimens with positive assurance.

However, an excellent opportunity to check the moisture content of the concrete was afforded in the summer of 1930 when a section of inspection tunnel 10 ft long was cut through old concrete near the base of the thrust-block by the plug-and-feather method. This concrete would not absorb additional

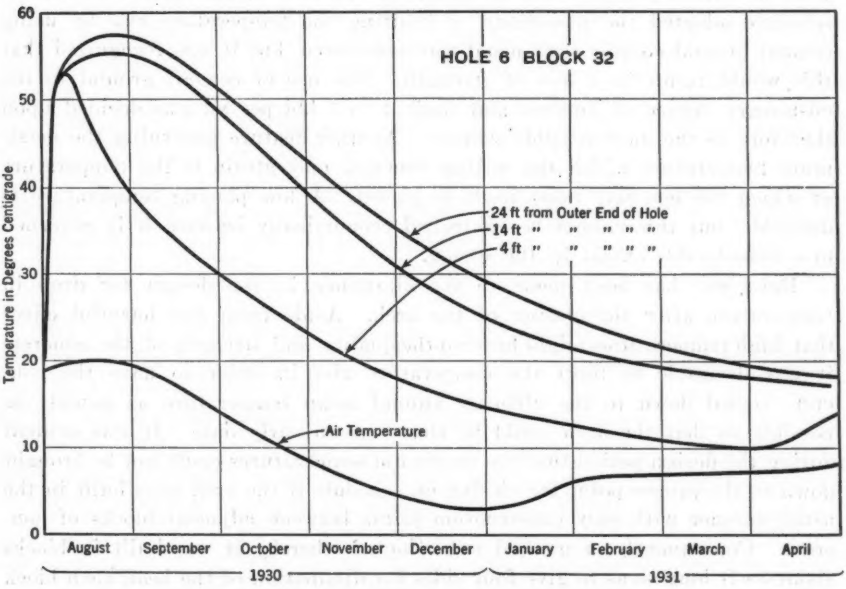


FIG. 12.—TEMPERATURES IN DIAMOND DRILL HOLES.

moisture, while the loss of moisture upon oven-drying was the same as for the cores. Tests for moisture content, made on samples taken from other parts of the dam, showed the same saturated moisture condition. The concrete in this dam was saturated to within 2 or 3 ft of the exterior surfaces which had been exposed for 8 to 12 months.

The average weight of the concrete, as determined from the cores, was 147.5 lb per cu ft as it came from the dam saturated with water and 138 lb after being oven-dried. The question of the actual weight of concrete in place in dams needs further investigation; it has a marked effect on design and means a real saving or waste of money, because weight will materially influence the cross-section of a dam. In securing cores from concrete dams it is desirable that larger ones be obtained in order to eliminate, as far as possible, the effect of the large pieces of aggregate. If possible, 6-in. cores should be recovered to approximate the diameter of the standard test cylinders.

HEATING OF CONCRETE AND EFFECT ON DESIGN OF DAM

It is well known that a mass of concrete, in setting, produces considerable heat. Opinion as to the effect of high temperatures on the quality and strength of concrete is varied. One of the principal items governing the

quantity of heat produced is the amount of cement used and the fineness to which it has been ground. In the case of the arch dam at Ariel the original specifications and designs called for 1.2 bbl of cement per yd. Test cylinders made from the cement and aggregate to be used in the arch indicated that 1 bbl of cement per yd would be sufficient. In view of the rapid construction schedule adopted the possibility of limiting the temperature rise by using cement ground coarser than usual was considered, but it was recognized that this would result in a loss of strength. The use of cement ground to the customary degree of fineness and limited to 1 bbl per yd was decided upon therefore as the most feasible scheme. Another feature governing the maximum temperature which the setting concrete may attain is the temperature at which the concrete mass itself is placed. A low placing temperature is desirable, but this cannot be controlled economically because it is governed to a considerable extent by the season.

Reference has been made to the allowance in the design for drop in temperature after the closing of the arch. Aside from any harmful effect that high temperatures might have on the quality and strength of the concrete it was desirable to limit the temperature rise in order to have the concrete cooled down to the ultimate annual mean temperature as quickly as possible so that the arch could be closed at an early date. It was evident during the design period that the maximum temperatures could not be brought down to the proper point for closing on schedule if the arch were built in the usual manner with only construction joints between adjacent blocks of concrete. Consequently, a method was adopted whereby it was built in blocks about 30 ft long so as to give four sides for dissipation of the heat, each block being separated from the adjacent block by a 2-ft open slot. To determine the value of cooling pipes, two of the blocks, namely, 27 and 31, on the south side of the river were built with vertical pipes extending their full height, through which water was pumped to assist in cooling the concrete. (See Fig. 6 for location of blocks.)

Thermometers, Provisions for Cooling, and Tests of Effectiveness

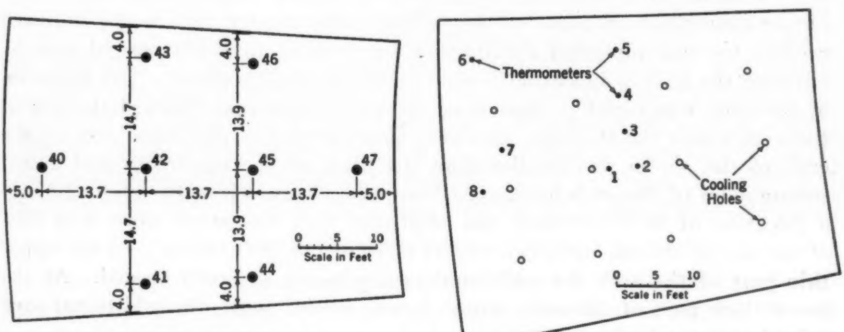
At Ariel a rather complete test program was undertaken to determine: (*a*) The maximum temperature in the concrete; (*b*) the rate at which internal heat would be radiated from uncooled blocks; and, (*c*) the rate at which internal heat would be radiated or removed by cooling water from artificially cooled blocks.

As mentioned previously two of the arch blocks, namely, 27 and 31, were provided with vertical pipes for circulating cooling water. It was desirable to have a complete picture of the maximum temperatures in the concrete and the rate of temperature drop in order to check the assumptions made in the design, and, therefore, to furnish reliable information for future designs of a similar character. The effectiveness of the cooling facilities in these two blocks furnished information as to the provisions that had to be made in the last three blocks of the arch. It was necessary to keep these three blocks

low until late in the spring of 1930 in order to help pass possible large floods. It was known that it would be necessary to build all three of them up rapidly and it was absolutely essential to reduce their internal temperature as much as possible in not more than two months' time.

Beginning with the placing of concrete in the arch in June, 1930, temperature readings were initiated in the thrust-block, and in a considerable number of the arch blocks. For this purpose resistance thermometers were embedded in the concrete and the leads were connected in groups to convenient terminal boards. Out of a total of 289 thermometers embedded all but 34 were in perfect working order at the time the arch was closed in April, 1931, and practically all of them were still in working order on March 1, 1932. Several of them were checked against mercury thermometers and the readings were found to agree within an average of 1° C. In computing the temperatures from the readings of the resistance thermometers allowance was made for the resistance of the leads as well as for the resistance of the connection to the test set.

Two methods of placing the resistance thermometers in a plane in a block were used. The first, shown in Fig. 13(*a*), was to place them in three parallel lines up stream and down stream across the block. The instruments on the middle one of these three lines were placed on either side of the center of the block. This method brought only two instruments near the center of the block, but provided a good arrangement for securing temperatures after the slots were filled. The second method (see Fig. 13(*b*)), was to place one



(*a*) BLOCK 25, AT ELEV. 90 (*b*) BLOCK 22, AT ELEV. 100

FIG. 13.—LOCATION OF RESISTANCE THERMOMETERS IN BLOCKS.

set of thermometers on a line extending up stream and down stream to the center of the block with another row at right angles to this, or lengthwise of the dam. In this case an additional thermometer was usually placed in a corner of the block about 5 ft from either face. This arrangement was not as satisfactory as the first one after the slots were filled and the water began to rise against the dam. However, the number of thermometers used was sufficient to overcome the deficiencies of either arrangement and to allow

consistent records to be obtained. Table 5 shows in what blocks and at what elevations the resistance thermometers were placed. It also shows in what blocks cooling holes were built.

TABLE 5.—LOCATION OF THERMOMETERS AND COOLING FACILITIES

Block No. (See Fig. 6)	Number of thermometers	Elevation of thermometers	Cooling equipment	Remarks
17...	26	210–190–170... 150–130–105...	....	Part of thrust-block
20...	19	210–160–120... Slot 25–26–32..	Slots to Elevation 100; twelve 5-in. cored holes..........	Rapid construction; raised 130 ft in 23 days
21...	22	210–150–110...	Twenty 5-in. cored holes; ten carried to top	Rapid construction; raised 142 ft in 14 days
22...	22	210–150–100*..	Twelve 5-in. cored holes; and one slot	Rapid construction; raised 152 ft in 23 days
25...	44	90–60–30*..... 5 and –40......	....	Highest block in dam; Elevation–73.7 to Elevation 240
26...	36	220–190–160... 130–100.......	....	....
27...	41	220–190–160... 130–100.......	Six 10-in. steel pipes.......	Very effective
31...	21	220–190–160...	One 12-in. steel pipe........	The first test block
32...	37	220–190–160... 130–100.......	....	A typical uncooled block
......	268	....	....	Total

* See Fig. 13.

It has been pointed out already that the fast construction schedule adopted for building the Ariel Dam required unusual steps to be taken in order that the arch might be closed without having temperatures appreciably higher than those adopted for the design. The greater part of the required cooling was accomplished by limiting the cement to 1 bbl per yd and by building the arch in blocks 30 ft long, with 2-ft slots between. The influence of the slots was variable, depending upon the elevation. Toward the top of the arch where the thickness was only about 20 ft the additional area at the ends of the blocks was smaller than the areas of the up-stream and down-stream sides of the arch block. At lower elevations, where the arch attained a thickness of 60 ft, or more, the additional area was much more than that of the two up-stream and down-stream faces of the 30-ft blocks. In the upper thin part of the arch the additional area was not so much needed. At the lower thick part of the arch, where it was needed most, the additional area gained was greatest.

Blocks 20, 21, and 22 (Fig. 6), which were left low through the fall and winter of 1930-31 to act as an emergency flood spillway, were raised one at a time as rapidly as possible. For example, Block 20 was raised 130 ft in 23 days; Block 21 was raised 142 ft in 14 days; and, Block 22 was raised 152 ft in 23 days.

It was known in advance that this would be the construction procedure, and experiments and observations in other blocks made it possible to determine the cooling facilities required in these three critical blocks. Vertical 5-in. cored holes were adopted. These holes were made by means of a 5-in., hollow, rubber conduit form which was raised at intervals as the concrete came up.

Block 31, well up on the left side of the arch, was built with one 12-in. cooling pipe up the center, and water was circulated through it continuously.

Block 27, near the left bank of the river, was equipped with six 10-in. pipes. Two of these pipes extended vertically from Elevation 90 to Elevation 170; two to Elevation 180, and the remaining two to the top of the block. These pipes were located in three rows of two each. The two pipes were 10 ft apart and the rows of pipes about 9 ft apart. The temperature records from Blocks 27 and 31 through which water was circulated continuously, gave information as to what could be accomplished by this method of cooling, using openings of different sizes and number. Fig. 14 shows one of the rubber conduits being raised.

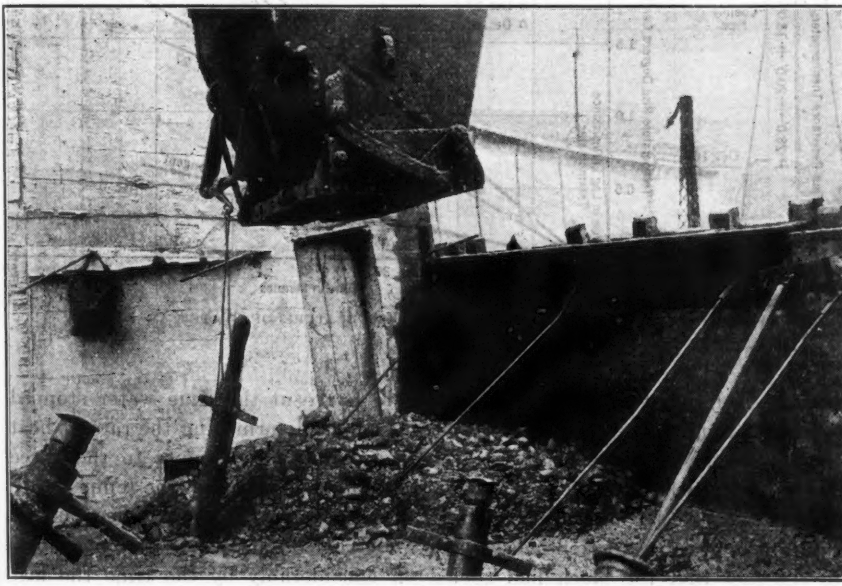


FIG. 14.—FIVE-INCH HOLLOW RUBBER CONDUIT BEING RAISED.

The circulating water was admitted to the bottom of the pipes, completely filled the vertical openings through the blocks, and escaped through overflows so that the heat was entirely removed by thermal conductivity without any reliance on evaporation. The possibility of cooling by the latter process was considered and might have been utilized by releasing a small quantity of water into the holes and inducing a natural or forced draft. It was thought at the time that this method would not be as effective as the one adopted and would be more difficult to regulate properly.

To determine the rate of heat transfer from the concrete to the water in the cooling openings the circulation was shut off in one of the blocks and the temperature at different depths in the hole was read by a series of resistance thermometers over an interval of time. The readings were continued until there was a notable decrease in the rate of temperature rise in the water.

Fig. 15 shows the arrangement of the 12-in. cooling pipe in Block 31, and Fig. 16, the temperature measurements made on three different dates. With water flowing in the cooling pipe at its normal rate, temperatures of the water were measured at the locations of the four resistance thermometers (Fig. 15). Then, with the water turned off (no overflow) time-temperature readings were

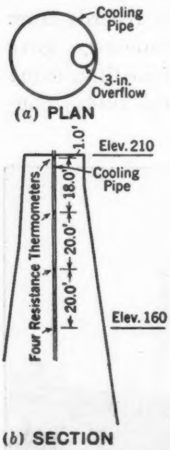


FIG. 15.—ARRANGEMENT OF 12-INCH COOLING PIPE IN BLOCK 32.

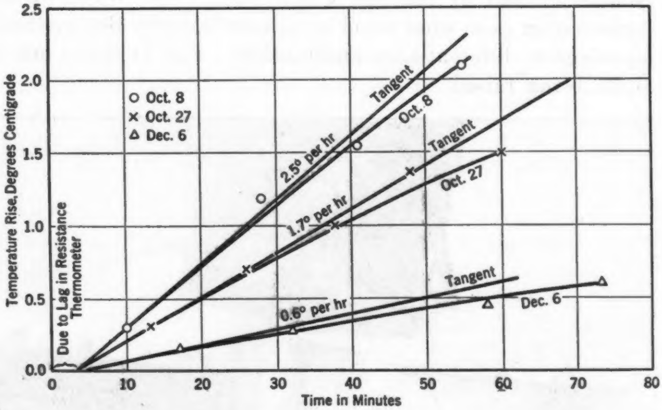


FIG. 16.—RISE OF TEMPERATURE OF WATER IN HOLES.

taken. A tangent to the time-temperature curve at the time water stopped flowing through the cooling pipe gives a means of determining the rate of heat flow from the concrete to the cooling pipe; that is, tangents to the three curves (Fig. 16) at the point of origin show the rate at which the temperature of the water rises. It will be noticed that there was a marked decrease in the rate for readings taken subsequent to October 8. This was due, no doubt, to the fact that the block had cooled off considerably by the time the last readings were taken on December 6. Tests similar to this one were made in the other cooling holes and, from the data thus obtained, a diagram was made to show the amount of heat removed, in kilogram-calories per hour per foot of hole. These results are shown by Fig. 17.

The readings obtained were not strictly comparable as the cooling holes passed through concrete of different ages so that the temperature gradients to the holes were not the same. Allowance has been made for these variables, however, so that the curves probably represent the rate of cooling that could be expected from the number and sizes of openings shown when used in a manner similar to that at Ariel Dam. It will be noted that the larger pipes removed considerably more heat than the smaller ones for the same difference in temperature between the cooling water and the maximum temperature of the block. As a result of these experiments, Block 20 was built with twelve 5-in. cored holes through it, Block 21 with twenty 5-in. holes (of which ten

only extended to the top), and Block 22 with twelve 5-in. holes. It will be recalled that these were the blocks to be built last and that they were raised 130 to 152 ft in 14 to 23 days. The actual reduction in the temperature of the concrete, accomplished by circulating water through the openings in the various blocks, is shown by Figs. 18 and 19.

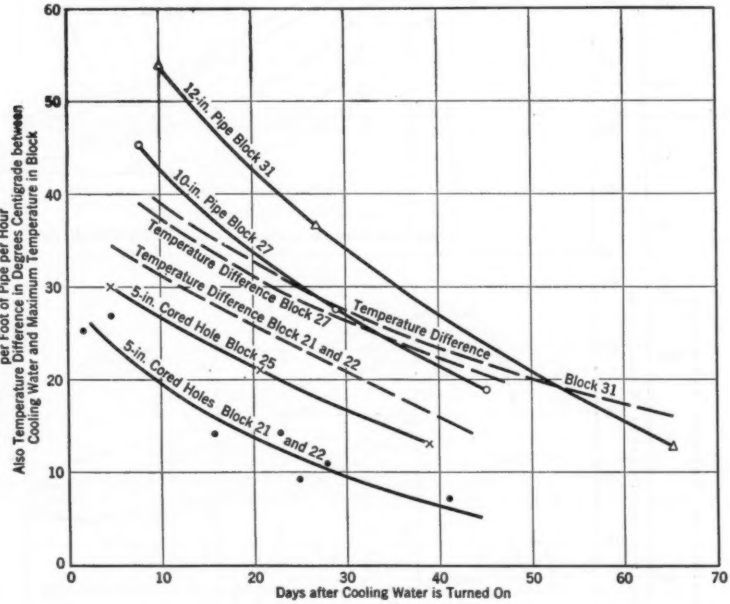


FIG. 17.—RATE OF HEAT REMOVAL BY COOLING WATER.

Referring to Fig. 19(*a*): Block 21 contained twenty 5-in. cored cooling holes and was built without slots; Block 27 contained six 10-in. cooling pipes; Block 25 had a slot (on one side only), at Elevation + 5; at Elevation − 40, Block 25 had no slots, but is confined between rock walls; Block 32, at Elevation 100, is affected by its proximity to the foundation rock; and all other blocks are normal.

Referring to Fig. 19(*b*): Block 31 contained one 12-in. cooling pipe at the center; Block 27 contained six 10-in. cooling pipes; and Block 21 contained twenty 5-in. cored holes, and was placed between Blocks 20 and 22 without slots.

Referring to Fig. 19(*c*): Block 27 contained two 10-in. cooling pipes, and Block 31 contained one 12-in. cooling pipe.

In addition to the artificial cooling of a few of the blocks by circulating water through them, all surfaces of the arch, as well as those of the remainder of the dam, were sprinkled continuously with water. Evaporation of this water assisted somewhat in the removal of heat from the concrete and, to a large extent, prevented surface checking and cracking.

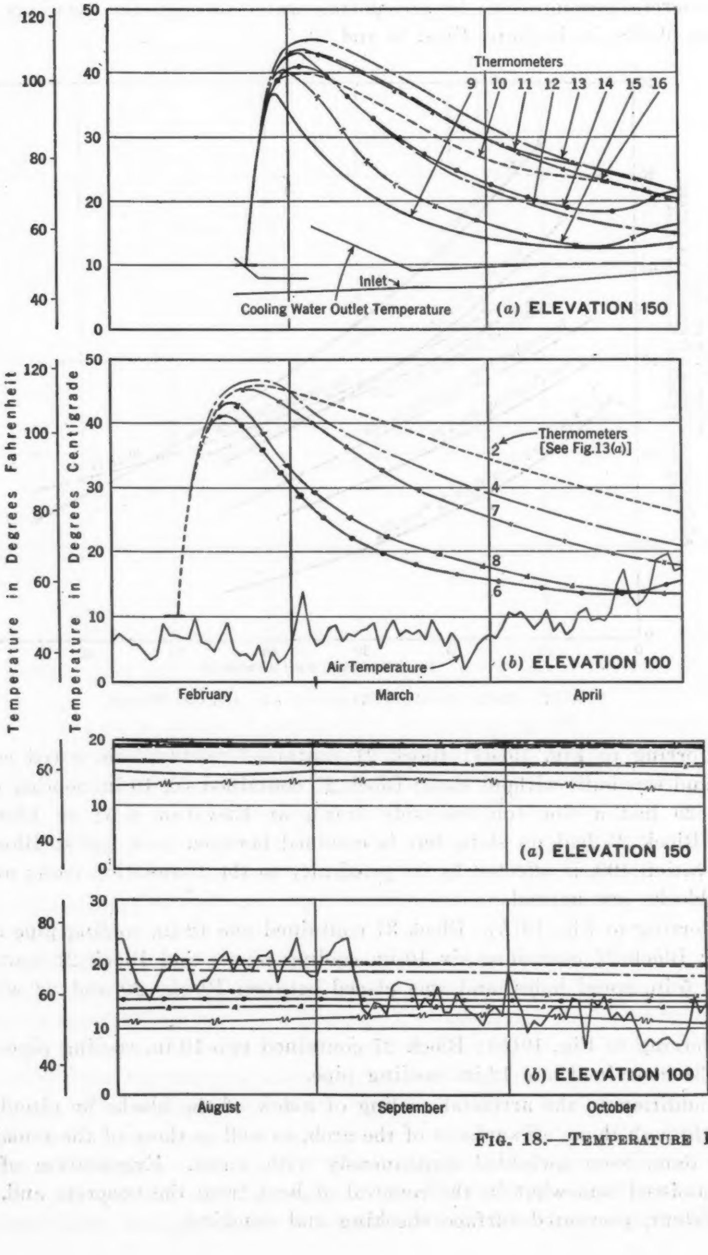
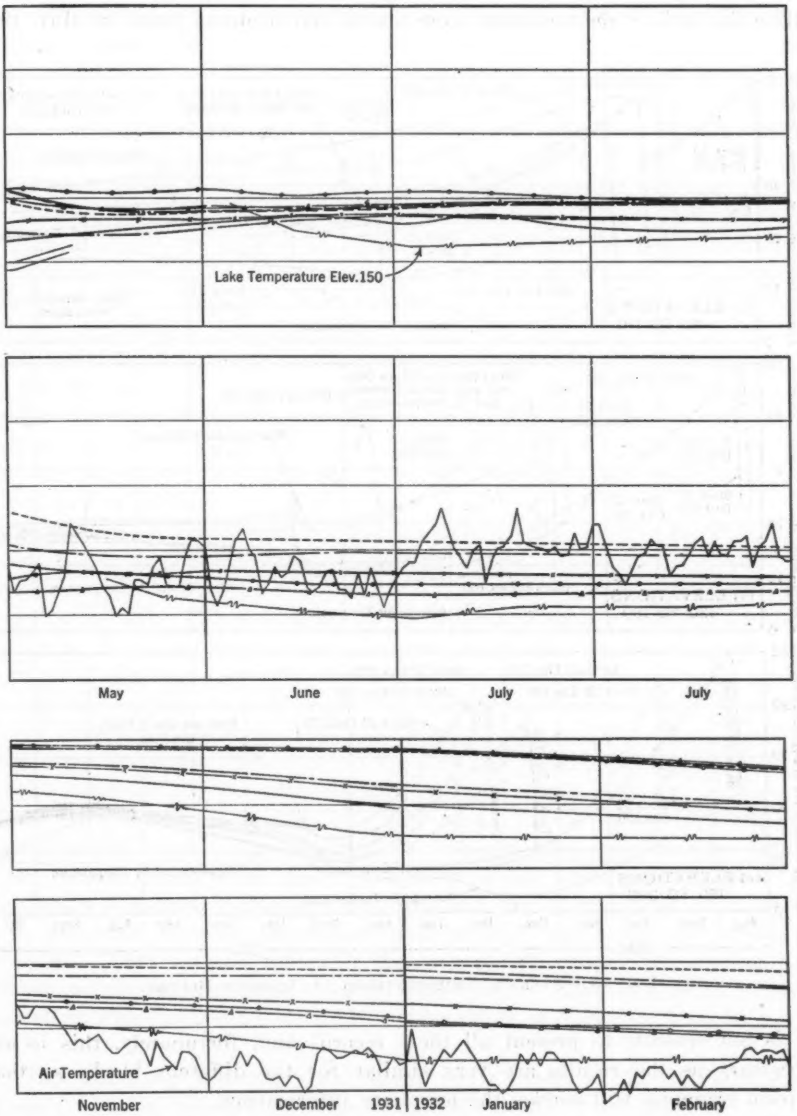


FIG. 18.—TEMPERATURE RECORDS



FOR BLOCK 22 (ARTIFICIALLY COOLED).

Temperatures in Typical Arch Blocks

The temperatures developed in the thrust-block and also in the arch blocks are recorded on seventy drawings which were kept up to date from the time the earliest thermometers were placed and readings began in May, 1930.

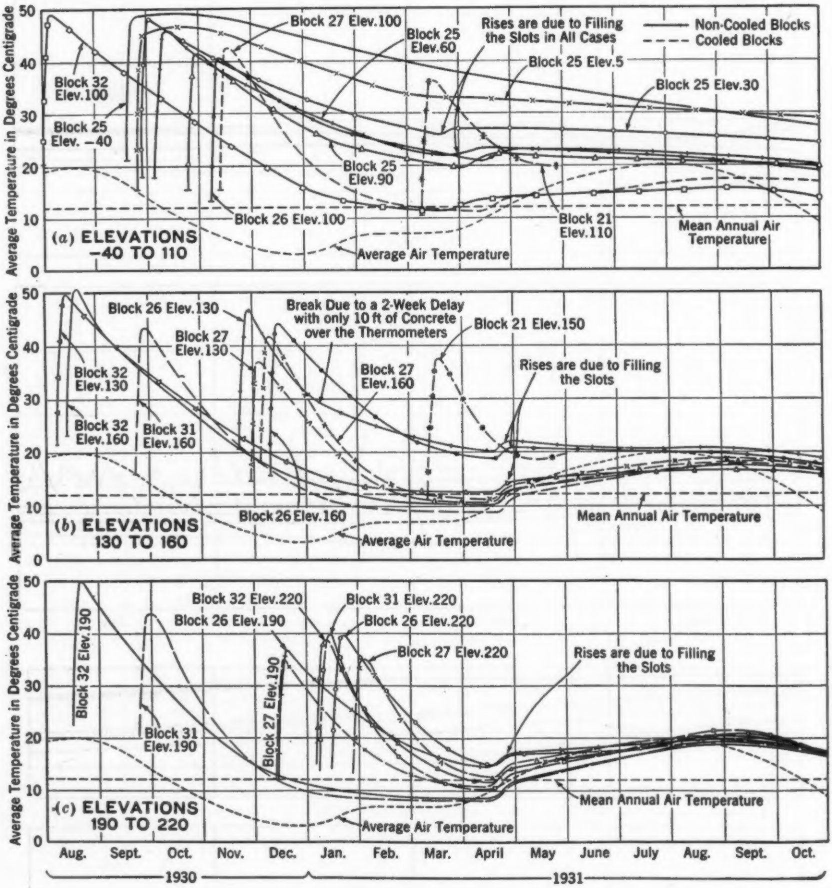


Fig. 19.—Average Temperatures in Various Blocks.

It is not feasible to present all these records and, fortunately, this is not necessary as the results are very similar for the different blocks so that typical examples will convey the necessary information.

The 289 electrical resistance thermometers previously mentioned were placed in the thrust-block and in nine of the arch blocks. This put an average of twenty-nine thermometers into a block, located in three to six horizontal planes.

The first thermometers were placed in the lower part of the thrust-block and readings were begun about May 1, 1930. The next thermometers were placed at five different elevations in Block 32 located near the south end of the dam and readings were begun about August 1, 1930. Another set of thermometers was placed in Block 31 which was built with the one 12-in. cooling hole. The readings from thermometers in this block began on September 30, 1930. Thermometers were placed in Block 27 in which the six 10-in. cooling pipes were placed, and readings were begun on November 12, 1930. Thermometers were placed in the lower part of Block 25 and readings were begun on September 15, 1930. In Block 26, thermometers were placed at five different elevations, and readings were begun on November 20, 1930. In each of Blocks 20, 21, and 22, which were built with the 5-in. cored holes for artificial cooling, three sets of thermometers were placed at different elevations, and readings were begun in the early part of February and March, 1931.

Records of the concrete temperatures in the uncooled Block 25 are given by Fig. 20, which shows the temperatures at Elevation 90, from August 1, 1930, to March 1, 1932. This example is typical of the results obtained in the uncooled blocks. No readings were possible on Thermometers 40, 41, and 43, probably because of broken leads. The positions of the thermometers are shown in Fig. 13(*a*).

Fig. 18 shows the temperatures at Elevations 100 and 150 in artificially cooled Block 22 from the time the readings began on February 12, 1931, to March 1, 1932. As noted, this block was built rapidly and had twelve 5-in. cored holes in it. The results shown for this block are typical of the results in Blocks 20 and 21 which were also built up rapidly at approximately the same time, and were also artificially cooled.

The temperature data obtained from the thermometer readings in all the blocks have been used to compute the "average" temperature of each block at each elevation where thermometers were placed. These "average" temperatures, extending from the time readings were begun in each case until November 1, 1931, are shown by Fig. 19. The curves for the uncooled blocks are shown by full lines, while those for the cooled blocks are shown by dashed lines. These curves show clearly the story of the rapid gain in heat, the maximum temperatures reached, and the slow but varying loss of this heat with the lapse of time, depending upon whether the block was cooled or uncooled. There is also indicated the average air temperature during this period and the date on which the slots were filled. These three sets of curves, as well as Figs. 18 and 20, show plainly the effect of filling the slots in April, 1931.

The following statements are based on a study of these temperature records, some of which extend over a period of twenty-two months:

1.—Rapid placing of the concrete, particularly in thick layers, caused high temperatures.

2.—The maximum temperature was 61.9° C (143.4° F).

3.—The highest placing temperature was 23.3° C (74° F).

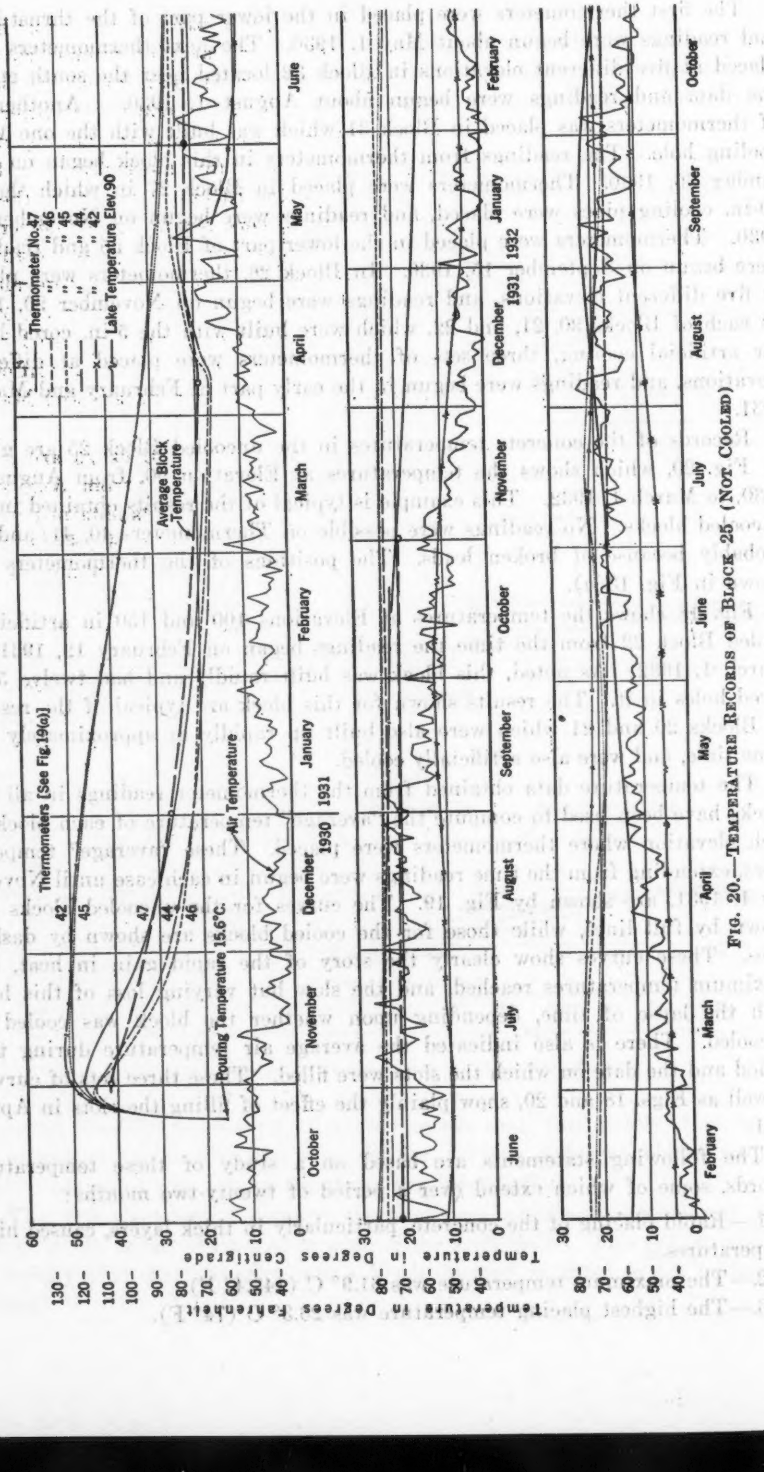


FIG. 20.—TEMPERATURE RECORDS OF BLOCK 25 (NOT COOLED)

4.—The highest "average" temperature of a lift was 51.0° C (123.8° F) for an uncooled block.

5.—The usual temperature without cooling was about 50° C (122.0° F).

6.—The lowest "average" temperature of a lift was 35.0° C (105.0° F) for the block cooled with six 10-in. pipes.

7.—The average rise in temperature was 23° to 28° C (73.4° to 82.4° F).

8.—Due to favorable radiation conditions the concrete within 4 ft of exposed surfaces never attained a high temperature.

9.—Temperature strains due to the unequal cooling of the exterior and interior of the concrete are severe enough to cause surface cracking of a large block if it is not reinforced, or unless the interior of the block is cooled artificially.

10.—Cored holes or a system of pipes through which cooling water can be circulated are practicable from a construction standpoint and are very effective in reducing the heat of the concrete if speed of construction is desirable.

11.—In spite of the high-speed construction schedule the temperature of the various blocks was satisfactory at the time of closing the arch in April,

TABLE 6.—ACTUAL TEMPERATURES IN ARIEL DAM AT TIME OF CLOSING, IN DEGREES CENTIGRADE

Block No.	18	19	20	21	22	23	24	25	26	27	28
Elevation:											
220	10.7	11.2	18.0	20.0	17.0	13.5	15.0	14.9	15.0	12.7	14.8
190	10.7	11.2	18.0	21.0	17.3	14.8	14.4	14.4	14.2	11.0	13.4
160	13.0	14.0	18.0	23.0	18.0	18.2	18.4	18.9	18.7	11.0	17.3
130	12.7	12.6	21.0	25.0	18.5	20.8	20.4	20.3	20.1	10.4	19.5
100	12.7	12.7	16.0	19.0	19.0	20.0	20.0	20.6	20.5	11.2	20.0
90	....	....	....	18.0	19.0	19.6	19.0	19.6	19.3	11.2	19.0
60	....	....	....	11.5	11.0	21.6	13.0	21.6	13.0	23.0	12.0
30	....	....	....	....	....	26.0	26.0	26.0	26.0	....	....
5	....	....	....	....	....	....	32.0	33.0	31.0	....	....
−40	....	....	....	....	....	....	....	38.5	....	....	....
Block No.	**29**	**30**	**31**	**32**	**33**	**34**	**35**	**36**	**37**	**38**	
Elevation:											
220	11.9	11.8	10.7	11.7	11.6	8.7	8.7	8.5	8.7	8.7	
190	9.8	8.7	8.0	8.6	8.6	8.7	8.5	8.5	8.6	....	
160	12.7	10.4	8.6	10.2	10.2	10.0	10.0	10.0	....	....	
130	14.8	12.7	12.7	12.7	12.7	11.0	11.0	11.0	....	....	
100	14.5	12.4	12.7	11.4	11.4	11.4	11.4	....	....	....	
90	14.8	12.7	12.7	....	....	....	....	....	....	....	
60	14.8	....	....	....	....	....	....	....	....	....	
30	....	....	....	....	....	....	....	....	....	....	
5	....	....	....	....	....	....	....	....	....	....	
−40	....	....	....	....	....	....	....	....	....	....	

1931 (see Table 6). For several different elevations the "average" temperature conditions (see Table 7) were, as follows:

From Elevation 240 to Elevation 190 the temperature of the concrete ring was down to about the average annual temperature (12° C).

From Elevation 190 to Elevation 60 the temperature was not more than 5° C above the average annual temperature.

From Elevation 60 to Elevation − 73 the temperature was considerably higher than the design value, but in this zone the arch is short and functions more nearly like a wedge.

TABLE 7.—TEMPERATURE DROPS AFTER CLOSING THE ARCH, IN DEGREES CENTIGRADE

Elevation	Average temperature of arch at closing, April 18, 1931	Drop in temperature to March 1, 1932	Drop in temperature below annual mean for which dam was designed
220	12.7	9.3	6.1
190	12.1	7.4	5.2
160	14.5	5.7	4.2
130	16.0	4.0	3.3
100	15.7	2.2	2.2
90	17.0	1.8	2.0
60	15.7	0.3	1.7
30	26.0	...	1.6
5	32.0	...	1.4
−40	38.5	...	1.3

The relative economy of building the dam with construction joints only and with embedded cooling pipes, as compared to building it as a series of blocks with slots between, is somewhat debatable. At Ariel, the Construction Organization was inclined to believe that the first plan would have been cheaper.

Due to the arrangement of slot construction the longitudinal shrinkage of the arch after closure was extremely small.

In connection with the temperature records it should be noted that the outside surfaces exposed to the sun were kept wet constantly by sprinkling. This sprinkling brought the temperature down about 5° C, or to about air temperature. Due to evaporation those parts of the surface in the shade had a temperature slightly lower than air temperature. The records indicate that the residual setting heat will not be entirely dissipated for two or three years.

It may be noted that the concrete temperatures began to go down again about October 1, 1931, following the high rise during the summer months which commenced with the filling of the slots in April. These temperatures follow nearly parallel to average daily air and reservoir water temperatures.

DEFLECTIONS OF THE ARCH DAM

Actual measurements of the deflections of full-sized arch dams are few. Observations of this type have been made on the Stevenson Creek Test Dam.[2] Concrete models of the Stevenson Creek and Gibson Dams have been tested, the latter in connection with the design of the Hoover Dam. A plaster-celite model of the Hoover Dam itself was tested in 1932.[3]

For the purpose of determining the deflection of the Ariel Arch twenty-one points were set in the arch and gravity abutment sections. Measurements were made: (1) After the slots were filled and just before water loading began; (2) at intervals during the time the reservoir was filling; and (3) at monthly intervals during 1931 and bi-monthly intervals during 1932, to determine the maximum deflection and the seasonal variation therein.

[2] *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3.

[3] "Model Tests Confirm Design of Hoover Dam," by J. L. Savage and Ivan E. Houk, Members, Am. Soc. C. E., *Engineering News-Record*, April 7, 1932.

The Ariel Arch was never subject to its water load until it was completed. This is a highly important consideration if stresses are to be expected, in different parts of the arch, in accordance with the calculated values.

The targets were brass plugs set into the concrete. The pointed, projecting part was painted with white enamel, and $\frac{3}{8}$-in. blackened holes on each vertical face were used as sights. Fig. 6(*b*) shows the location of the various deflection points on the arch and thrust-block. The deflections were read by means of horizontal angles between fixed targets and the arch targets, from an instrument base on either side of the river. These two bases were tied together by a triangulation system carefully laid out between them and a base line on the right bank of the river (see Fig. 21).

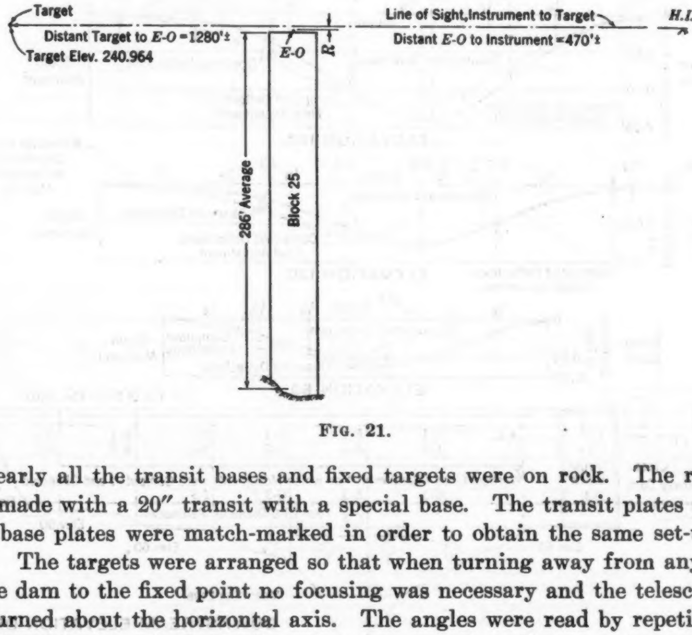


FIG. 21.

Nearly all the transit bases and fixed targets were on rock. The readings were made with a 20″ transit with a special base. The transit plates and the fixed base plates were match-marked in order to obtain the same set-up each time. The targets were arranged so that when turning away from any target on the dam to the fixed point no focusing was necessary and the telescope was not turned about the horizontal axis. The angles were read by repetition, 10 to 20 hr being required to take a complete set of readings on the twenty-one points. The measurements taken on May 11 and 12, 1931, just before the reservoir began to fill, were considered the "zero" readings. Fig. 22 shows the radial, and Fig. 23, the tangential, deflections for various dates to July 15, 1932.

The computed deflections are for water at Elevation 240; they have been increased to include the deflection of the thrust-block. The dam was closed in May, 1931, and the readings taken at that time are considered as the zero readings or reference points for all future readings. Deflections were read monthly during 1931 and every two months during 1932. Only four sets of these readings are shown in Figs. 22 and 23. Maximum deflections occurred in May, 1932, and subsequent deflections decreased as the temperature of the

structure increased. In Fig. 23 clockwise displacements indicate movement toward the south abutment and counter-clockwise displacements indicate movement toward the thrust-block.

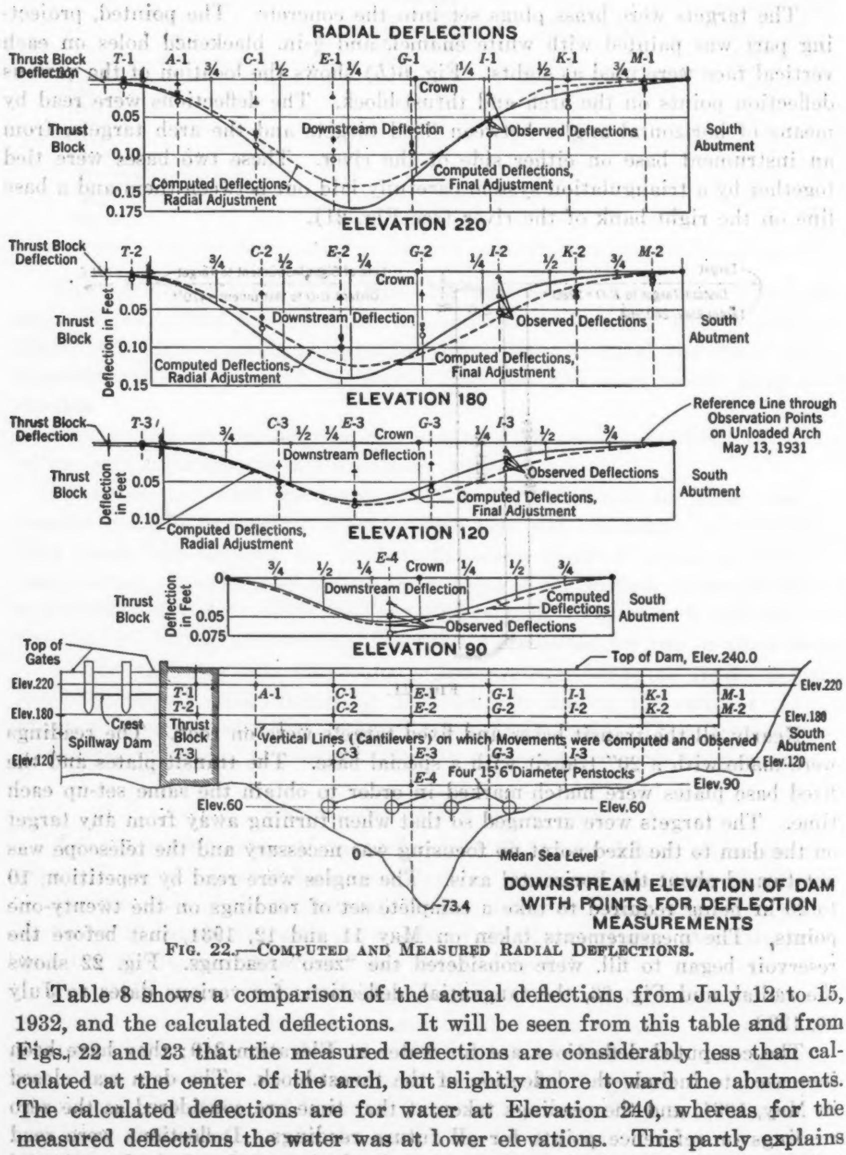


FIG. 22.—COMPUTED AND MEASURED RADIAL DEFLECTIONS.

Table 8 shows a comparison of the actual deflections from July 12 to 15, 1932, and the calculated deflections. It will be seen from this table and from Figs. 22 and 23 that the measured deflections are considerably less than calculated at the center of the arch, but slightly more toward the abutments. The calculated deflections are for water at Elevation 240, whereas for the measured deflections the water was at lower elevations. This partly explains why they have not as yet reached their maximum. Another reason is that the concrete had not yet reached the design temperature, except above Elevation 190 (see Table 6). A third reason is the fact that the modulus of

elasticity of the foundation and abutment rock probably is not exactly the same as that assumed in the design. The maximum yearly deflection is expected to occur about the last of April of each year, as this is the time when

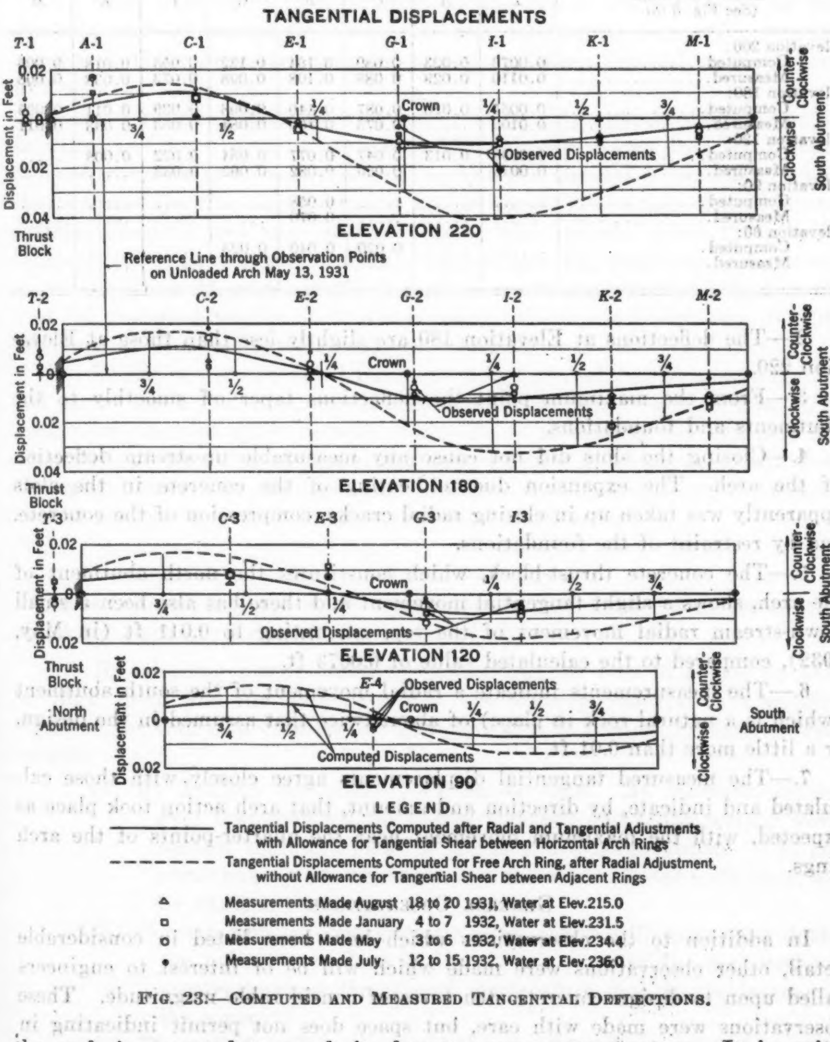


FIG. 23.—COMPUTED AND MEASURED TANGENTIAL DEFLECTIONS.

the arch is expected to reach its lowest average temperature. It is quite probable that the maximum deflection will not be reached before April, 1933.

Recorded observations on the behavior of the arch for fourteen months (to July 15, 1932), indicate that:

1.—The maximum recorded deflection (in May, 1932) is 0.108 ft (1.3 in.) at Elevation 220 in Block 25.

TABLE 8.—COMPARISON OF CALCULATED AND MEASURED RADIAL ARCH DEFLECTIONS

Deflection points (See Fig. 6 (*b*))	*T*	*A*	*C*	*E*	*G*	*I*	*K*	*M*
Elevation 200:								
Computed	0.0073	0.023	0.089	0.164	0.132	0.053	0.018	0.006
Measured	0.0110	0.029	0.088	0.108	0.098	0.073	0.038	0.008
Elevation 180:								
Computed	0.0055	0.020	0.087	0.140	0.098	0.039	0.011	0.003
Measured	0.0100	.....	0.075	0.100	0.084	0.053	0.033	0.011
Elevation 120:								
Computed	0.0038	0.013	0.047	0.077	0.054	0.022	0.008	.....
Measured	0.0010	.....	0.069	0.082	0.062	0.033	.....	.....
Elevation 90:								
Computed	......	.....	.....	0.058	.....	.....	.....	.....
Measured	......	.....	.....	0.070	.....	.....	.....	.....
Elevation 60:								
Computed	......	.....	0.020	0.040	0.023	.....	.....	.....
Measured	......	.....	.....	.....	.....	.....	.....	.....

2.—The deflections at Elevation 180 are slightly less than those at Elevation 220.

3.—From the maximum point the deflections taper off smoothly to the abutments and foundations.

4.—Closing the slots did not cause any measurable up-stream deflection of the arch. The expansion due to heating of the concrete in the slots apparently was taken up in closing radial cracks, compression of the concrete, and by restraint of the foundations.

5.—The concrete thrust-block, which constitutes the north abutment of the arch, shows a slight tangential movement and there has also been a small down-stream radial movement of the top, amounting to 0.011 ft (in May, 1932), compared to the calculated value of 0.0073 ft.

6.—The measurements indicate a radial movement of the south abutment (which is a natural rock in place) of about twice that assumed in the design, or a little more than 0.01 ft.

7.—The measured tangential displacements agree closely with those calculated and indicate, by direction and amount, that arch action took place as expected, with the maximum movement near the quarter-points of the arch rings.

GENERAL OBSERVATIONS

In addition to the observations which have been listed in considerable detail, other observations were made which will be of interest to engineers called upon to design concrete structures of considerable magnitude. These observations were made with care, but space does not permit indicating in detail the procedure that was followed:

1.—A test was made to determine whether the holes cored in the concrete for cooling purposes would cause radial cracking. An experimental block was tested and careful examination did not disclose any cracks and a light wire spiral showed no indication of change in electrical resistance which would have been the case if cracks of any size had developed.

2.—The specific heat of saturated concrete was found to be 0.26, while the specific heat of oven-dried concrete was 0.21. It is believed that the higher figure should be used in determining the heat released in a dam.

3.—Tests were made to determine the amount of heat liberated by the concrete after it was in the forms. It was determined that under average conditions 114 calories per gram of cement had been released at the end of 45 days.

4.—As a result of pouring some concrete in this arch dam with cooling facilities and some without cooling facilities, it was found that the placing of fresh concrete at a lower temperature had two distinct advantages: First, the temperature rise due to setting was so much slower that radiation and artificial cooling had time to take effect; and, second, since the placing temperature was lower the maximum temperature was correspondingly lower.

5.—In Block 25, which is 286 ft high, measurements indicated that in the first ten months after the block was completed there was a shrinkage of 0.046 ft compared to a calculated shrinkage of 0.031 ft.

6.—Facilities were provided for measuring hydraulic uplift on the base of the thrust-block and at three different horizontal days' work joints. The maximum observed uplift between the rock and the base of the dam was less than 20% of the static head and this was within 5 ft of the up-stream face. No uplift occurred on the days' work joints, indicating that water-tight joints can be secured by proper design and careful construction.

Conclusions

It is believed that the many facts determined from the extensive tests at the Ariel Dam warrant the following conclusions:

1.—By adopting proper construction methods an arch dam may be built at high speed and the temperature of the concrete reduced to a suitable figure at the time of arch closure. This end may be attained by the following means:

(*a*) By limiting the quantity of cement used. An excess of cement over that required for proper strength and quick setting works against the attainment of low concrete temperatures. The proportioning of the aggregate and the water-cement ratio must also be watched constantly.

(*b*) Construction in comparatively short blocks will more than double the radiating surface in the thicker parts of the arch and will greatly increase the rate of temperature drop in the interior of the concrete.

(*c*) The use of cooling pipes or holes cored in the concrete is entirely practicable from a construction standpoint, and the circulation of cold water through these openings will assist in materially reducing the concrete temperature by increasing the rate of heat removal. This artificial cooling will also aid in the prevention of surface cracks. Where artificial cooling is used it is essential that it is begun as soon as possible after the concrete is placed.

2.—Hollow rubber cores make excellent forms for cooling holes and they eliminate entirely the possibility of grout and débris collecting in the holes. Steel pipes are entirely suitable for forming the holes, but are not as convenient as the cores.

3.—Electrical resistance thermometers of a high degree of reliability can be obtained. These thermometers can be placed in the concrete and the leads can be brought out without interfering materially with the concreting work or adding to its cost.

4.—If necessary the concrete can be placed rapidly in large quantities and to a great depth without sacrificing any of its desirable qualities. In the Ariel Dam one block was built up 125 ft in 10 days. High temperatures in the interior of a block, 40 by 90 ft in plan, were not detrimental to the strength of the concrete.

5.—The temperature of the concrete in parts of the structure eighteen months after the construction began is still (1933) somewhat above the annual average air temperature, but is slowly approaching that figure. The maximum observed temperature in the concrete was 61.9° C (143.4° F) and the average temperature rise varied from 23° to 28° C (73.4° to 82.4° F).

6.—Diamond drill cores taken from the dam at several places indicated that a saturated condition of the concrete existed fifteen months after placing. These cores also indicated an increase of strength from the surface toward the interior, the concrete in the interior having a compressive strength as great as, or greater than, that indicated by the regular test cylinders of equal age.

7.—A measurable vertical shrinkage took place in the concrete and if it had not been for the slot construction there would have been a high concentration of loads at the quarter-points of the dam where the steep sides of the canyon occur. The vertical shrinkage subsequent to the closing of the arch has not caused the diagonal cracks which have occurred in some dams.

8.—There is an entirely satisfactory agreement between the actual and the calculated radial and tangential deflections, both in direction and in amount. Measurements show a down-stream movement of the top of the thrust-block of 0.0037 ft more than calculated.

9.—Annual temperature changes account for more than 30% of the deflection of the Ariel Arch Dam. If a drop in seasonal temperature has such an effect on the dam it is evident that the chemical heat developed in the concrete should be removed, as far as possible, in order to avoid additional deflection and stress.

10.—The leakage under the dam is extremely slight. This can be accounted for by the particular care taken in the preparation of the foundations and the careful grouting of them for water-tightness.

ACKNOWLEDGMENTS

The Ariel Development was designed by the Engineering Department of Electric Bond and Share Company and was built by Phœnix Utility Company. The late Mr. H. F. Lincoln was Local Construction Manager, and was assisted by R. C. Booth, M. Am. Soc. C. E., as Assistant Construction

Manager. The various temperature and deflection measurements were carefully carried out by R. W. Spencer, Assoc. M. Am. Soc. C. E., Resident Engineer, and I. L. Tyler, Assoc. M. Am. Soc. C. E., Assistant Engineer. Previous to the starting of design and construction, the project had been subject to study for some years under the direction of Lyman Griswold, M. Am. Soc. C. E., Consulting Engineer for the Northwestern Electric Company. D. C. Henry, Vice-President, Am. Soc. C. E., and Consulting Engineer, at Portland, Ore., represented the State of Oregon on design and construction matters in connection with the entire dam, and his wide experience was of value in considering the many special problems involved. L. T. Merwin, Vice-President and General Manager of Northwestern Electric Company, took an active interest in making it possible to secure the information presented in this paper and in making it available to the Engineering Profession.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

WIND STRESSES BY SLOPE DEFLECTION AND CONVERGING APPROXIMATIONS

By John E. Goldberg[1], Jun. Am. Soc. C. E.

Synopsis

The purpose of this paper is to present a method of wind-stress analysis for rigid frames, based on the well-known slope-deflection analysis. By adjustments in the formulas and manipulation of the computations, rapid successive solutions are effected which after a few computations approach final accurate results, consistent with previously accepted refined but arduous processes.

The method is to be considered as basic or fundamental, directly applicable both to the analysis of stresses in individual bents acted upon by known horizontal forces and to structures having symmetrical layouts. For the latter type of problem a conception of a "composite bent" is introduced as a tool for structural analysis. Finally, a solution is proposed for secondary wind stresses, caused by column strain. The proposed methods have the merits of accuracy, speed, and practicability.

Introduction

Since its original presentation the slope-deflection method of wind-stress analysis by G. A. Maney, M. Am. Soc. C. E.,[2] has been considered to be one of the best of the so-called "exact" methods. As first presented, however, it was too laborious for general use. Many variations of the original have since been proposed, designed to eliminate the tedious work necessary for a solution of the great number of simultaneous equations which form the basis of the analysis and at the same time to retain the theoretical accuracy. In addition, many admirable methods based on other theories have been proposed from time to time. Broadly speaking, however, because of elements of inaccuracy, impracticability, or lack of general application, no single method has enjoyed a widespread every-day use. The slope-deflection method

Note.—Discussion on this paper will be closed in September, 1933, *Proceedings*.

[1] Chicago, Ill.

[2] *Engineering Studies No. 1*, Univ. of Minnesota, 1915.

suffered, with other methods, because of its tediousness, but it has always been used as a standard by which the accuracy of other and newer methods may be measured.

A more or less recent consideration which has added to the complications of an already involved problem is the effect of column strains under wind load. This paper is believed to present a simple, and yet theoretically correct, method of analysis, based on the fundamental slope-deflection theory, which can be made to include, when desired, the effect of column strains.

General Scope and Considerations

The treatment covers the following cases: (1) Direct analysis is applied to an individual bent; (2) when the structural layout of the whole building is symmetrical or approximately so, and particularly when the tower is an integral part of the structure, that is, not carried on trusses, the structure as a whole may be analyzed by means of the conception of the "composite bent" as explained later; and (3) when the tower is supported above the main part of the building by trusses, some assumption regarding the degree of fixity of the lower connections of the columns at the bottom of the tower having been made, it is possible to analyze the tower as a separate structure in which the trusses are analogous to foundations.

Whether the symmetrical or asymmetrical layout is the more important is, of course, a matter of opinion and personal experience; but it appears that a large percentage of the more important skyscrapers are either symmetrical in layout or approach symmetry and may be analyzed on that basis. In view of the fact, however, that apparently no simple and yet rational method of analyzing an extremely asymmetrical building as a unit is available, it would seem best, in problems of this type, to divide the design load between the various braced bents on some arbitrary or empirical basis.

Notation

For convenience, the various symbols are here collected and defined. Clockwise moments and angles may be assumed to be either positive or negative, as long as the same convention is observed throughout any analysis. The use of consistent units is presupposed. In interpreting the definitions, the reader should bear in mind the conception of the composite bent, as described subsequently. Following is the notation:

f = the ratio, for any column, of its relative stiffness, K_c, to ΣK_c for the story;

h = story height = length of column;

m = end moment: m_{AB} = actual moment at the end, or Joint A of a member, AB; that is, for example, m_{nX-mX} = actual moment at the end, or Joint nX of the member, $nX - mX$; $m_{F\text{-}AB}$ = fixed-end moment at the end, or Joint A of the member, AB;

C = the ratio, for any member, of K to $\dfrac{2EI}{L}$, a constant for any analysis; it also equals the ratio of any rotation, in radians, to the same rotation in anomalous units;

D = relative transverse displacement of ends of member (horizontal for columns, vertical for beams and girders);
E = modulus of elasticity of the material of the members;
I = moment of inertia of the cross-section of the member;
K = relative stiffness of a member: K_{AB}, of the member, AB; K_i, of a member in which the rotation at the far end is θ_i; K_C, of a column; and K_G, of a girder (or beam):
L = length of a member (particularly of beams);
M = "wind moment" = horizontal shear on a story times the story height;
$R = \frac{D}{h}\left(\text{or } \frac{D}{L}\right)$ in the same anomalous units: R' = an initial or estimated value of R for any story;
S = rotation of a joint, in radians: S_A, of the joint, or end, A; and, S_B, of the joint, or end, B;
θ = rotation of a joint in anomalous units: θ_A, of Joint A; θ_B, of Joint B; θ', an initial, or estimated value of the average of all θ-values of any story; and θ_i, the rotation of a joint at the opposite end of a member that frames into the joint under consideration.

A "story" consists of the columns, together with the girders or beams immediately above. For properties or variables of the various stories, columns, joints, or beams, subscripts are used. For example, the number of the story, counted up from the foundation, is designated by a lower case letter, such as m, n, or o, in formulas and equations, or by a number, in the actual analysis. Columns, joints, and beams or girders are similarly designated by an upper case letter, for example, A, B, C, ... W, X, Y; for convenience, A is taken at the left. Thus, the Xth column of the nth story is designated as the nXth column; the third joint of the fourth story, as Joint $4C$; and the relative stiffness of the nXth column, as K_{CnX}. The relations of the elastic line of a distorted column are shown in Fig. 1.

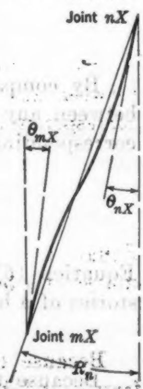


FIG. 1.—COLUMN, DISTORTION.

FUNDAMENTAL EQUATIONS

For simplicity, the following theory is developed on the basis of the single bent, either individual or composite. The general equation on which the slope-deflection method of analysis is based is the expression for the moment at the end, A, of the member, AB:

$$m_{AB} = m_{F\text{-}AB} + \frac{2EI}{L}\left(3\,\frac{D}{L} - 2\,S_A - S_B\right) \qquad (1)$$

in which, $\frac{2EI}{L}$ is the stiffness factor. In this form it includes the effects of: (1) Beam loading; (2) joint translation; and (3) joint rotation.

In the usual conception and handling of wind-stress problems, the designer is not generally concerned with beam loading; he dissociates vertical (dead and live) load and wind load, and also assumes that all loads are applied at

the panel points. Thus, he eliminates the term, m_{F-AB}, from this particular application of the fundamental equation.

As a means of reducing the work to simpler figures, there is introduced into the formula the idea of relative stiffness, substituting K for $\frac{2EI}{L}$, and θ and R (which are in anomalous units) for the corresponding S and $\frac{D}{L}$ (which are in radians). Then, for the Xth column of the nth story:

$$m_{nX-mX} = K_{CnX}\,(3R_{nX} - 2\,\theta_{nX} - \theta_{mX}) \quad \text{.............(2)}$$

and for the girder (or beam) which lies between the joint, nX, and the joint, nY, neglecting the effect of column strains:

$$m_{nX-nY} = -K_{GnX}\,(2\,\theta_{nX} + \theta_{nY}) \quad \text{................(3)}$$

Where column strains, of known or assumed amount, are taken into account as causing relative transverse displacement of the ends or joints, the most convenient form is:

$$m_{nX-nY} = \frac{6EI}{L}\,\frac{D}{L} - K_{GnX}\,(2\,\theta_{nX} + \theta_{nY}) \quad \text{...........(4)}$$

By comparing Equations (1) and (2), it may be seen that the relation between any horizontal deflection (occurring in one column length) and its corresponding, R, is:

$$D = Rh\,\frac{K}{\frac{2EI}{h}} = CRh \quad \text{......................(5)}$$

Equation (5) is of use in calculating the actual deflections of the various stories of a building after the various R-values have been obtained.

The Bent Equation

Because the ratio by which the total wind load is divided between the frame, and the walls, floors, etc., is an unknown and not easily estimated quantity, it is usual to assume that the frame carries the entire load, giving, in effect, an increased factor of safety or factor of reduction for both stresses and deflections.

Consider a single bent as being acted upon by known horizontal forces, and cut all the columns of the nth story by a horizontal plane. Then, applying the law of statics, $\Sigma H = 0$, to the part of the bent above the cut section,

$$\Sigma\ \text{(column shears)} = \text{external shear}$$

in which, the external shear is the total of all loads above the cutting plane. Multiplying both sides of the equation by h, the story height,

$$\Sigma\ \text{(column shears)}\ (h) = \text{(external shear)}\ (h) = M_n$$

Since shear is the rate of change of moment,

$$\Sigma\ \text{(both end moments of all columns)} = M_n$$

By Equation (2), the sum of both end moments of the ${}_{n}x th$ column is,

$$K_{cnx}\,(6\,R_{nx} - 3\,\theta_{nx} - 3\,\theta_{mx})$$

and for all columns of the nth story, since R_n is the same for all when they are of a uniform height,

$$6\,\Sigma\,K_{Cn}\,R_n - \Sigma\,[K_{CnX}\,(3\,\theta_{nX} + 3\,\theta_{mX})] = M_n \,\ldots\ldots\ldots\ldots (6)$$

This becomes the standard form of the bent equation.

The Joint Equation

To each joint the law of statics, $\Sigma\,m = 0$, may be applied. Take the joint, nX, as typical. Then,

$$m_{nx\text{-}nw} = -\,K_{Gnw}\,(2\,\theta_{nx} + \theta_{nw})$$

$$m_{nx\text{-}ny} = -\,K_{Gnx}\,(2\,\theta_{nx} + \theta_{ny})$$

$$m_{nx\text{-}mx} = K_{cnx}\,(3\,R_n - 2\,\theta_{nx} - \theta_{mx})$$

$$m_{nx\text{-}ox} = K_{cox}\,(3\,R_o - 2\,\theta_{nx} - \theta_{ox})$$

Hence, by addition of these four equations,

$$\Sigma\,m = 3\,K_{cnx}\,R_n + 3\,K_{cox}\,R_o - 2\,\Sigma\,K\,\theta_{nx} - \Sigma\,(K_i\,\theta_i) = 0 \ldots (7)$$

in which, $\Sigma\,(K_i\,\theta_i)$ is the summation of the products of the K-values for each member framing into the joint under consideration times the θ-values for the joints at the respective far ends of those members. Equation (7) is the general form of joint equation as used in primary wind-stress analysis.

Methods of Solution

The original method of solution consisted of writing a bent equation for each story and a joint equation for each joint, solving these equations simultaneously for the various R and θ-values. The R and θ-values thus obtained were substituted in expressions which had the form of Equations (2) and (3), to determine the correct end moments throughout the bent. Obviously, for any but the simplest bents, the labor involved in such a solution would be prohibitive.

It is well known that, under wind load, the points of contraflexure, particularly of columns, tend to approach the mid-lengths of members. The method to be outlined herein takes due account of this action; but, by using it only to obtain initial values of the various R's and initial average values of the θ's of each story, it does not, so to speak, place too much confidence in this fact.

Taking the part of the bent adjacent to the mth story, and assuming all joints to have the same rotation, θ'_m, the bent equations for the mth and nth stories become,

$$M_m = \Sigma\,K_{cm}\,(6\,R_m - 6\,\theta'_m)$$

$$M_n = \Sigma\,K_{cn}\,(6\,R_n - 6\,\theta'_m)$$

or, by addition,

$$M_m + M_n = 6 \Sigma K_{Cm} R_m + 6 \Sigma K_{Cn} R_n - 6 \theta'_m (\Sigma K_{Cm} + \Sigma K_{Cn}) \ldots (8)$$

The joint equation for any joint of the mth story becomes,

$$3 K_{cm} R_m + 3 K_{cn} R_n - 3 \Sigma K \theta'_m = 0$$

and for all joints of the mth story, taken together:

$$3 \Sigma K_{cm} R_m + 3 \Sigma K_{cn} R_n - \Sigma (3 \Sigma K) \theta'_m = 0 \ldots\ldots\ldots\ldots (9)$$

Eliminating R_m and R_n between Equations (8) and (9):

$$\Sigma (6 \Sigma K) \theta'_m = M_m + M_n + 6 \theta'_m (\Sigma K_{cm} + \Sigma K_{cn})$$

Since $\Sigma (6 \Sigma K) = 12 \Sigma K_{Gm} + 6 \Sigma K_{Cm} + 6 \Sigma K_{Cn}$, this equation reduces to:

$$\theta'_m = \frac{M_m + M_n}{12 \Sigma K_{Gm}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

This gives a formula for the initial average values of the joint rotations in terms of known quantities.

It is customary to assume a degree of fixity for the columns at the foundations, generally either pin-ended or fully fixed, and for these conditions the foregoing derivation may be modified to obtain an expression for θ'_1. For the pin-ended condition, Equation (10) becomes:

$$\theta'_1 = \frac{2 M_1 + M_2}{12 \Sigma K_{G1}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (10a)$$

and for the fully fixed condition:

$$\theta'_1 = \frac{M_1 + M_2}{12 \Sigma K_{G1} + \Sigma K_{C1}} \ldots\ldots\ldots\ldots\ldots\ldots (10b)$$

Proceeding to a simplified conception of the bent equation, the standard Equation (6) for R_n is solved to obtain:

$$R_n = \frac{M_n}{6 \Sigma K_{Cn}} + \frac{\Sigma [K_{CnX} (\theta_{mX} + \theta_{mX})]}{2 \Sigma K_{Cn}} \ldots\ldots\ldots (11)$$

which may be written in the form:

$$R_n = \frac{M_n}{6 \Sigma K_{Cn}} + \text{(weighted average value of all } \theta_m\text{'s and } \theta_n\text{'s)} \ldots\ldots (11a)$$

For calculating an R-value from any series of θ-values in an actual analysis, the following is perhaps the best form:

$$R_n = \frac{M_n}{6 \Sigma K_{Cn}} + \Sigma \left[f_{nX} \frac{(\theta_{mX} + \theta_{nX})}{2} \right] \ldots\ldots\ldots\ldots (11b)$$

in which, f_{nx} is the ratio, for the Xth column of the nth story, of its K-value to ΣK_{cn}. For the initial value of R_n, in terms of initial θ-values:

$$R'_n = \frac{M_n}{6 \Sigma K_{Cn}} + \frac{\theta'_m + \theta'_n}{2} \ldots\ldots\ldots\ldots\ldots\ldots (11c)$$

The most convenient form of the joint equation is obtained by solving Equation (7) for its corresponding θ:

$$\theta_{nx} = \frac{3\,K_{Cn\,x}\,R_n + 3\,K_{Co\,x}\,R_o - \Sigma\,(K_i\,\theta_i)}{2\,\Sigma\,K} \qquad \text{(12)}$$

Equations (10) (and (10*a*) or (10*b*)), and (11*c*), therefore, are of use in estimating the θ and R-values of each story. Equations (12) and (11), (11*a*), or (11*b*) are used to determine more nearly correct θ and R-values, and, if the work is carried far enough, absolutely correct values may be obtained from these equations.

Order of Procedure

The recommended procedure in a primary wind-stress analysis may now be outlined. Each consecutive step is numbered for convenience. The same numbering is applied to the illustrative problem shown in Fig. 2 and explained subsequently. The procedure is as follows:

(1) Calculate the relative stiffness of each member of the frame; also, calculate $\Sigma\,K_G$ and $\Sigma\,K_C$ for each story, $\Sigma\,K$ for each joint, and f for each column.

(2) Determine the horizontal shear on each story, and from this determine M = shear times story height for each story.

(3) Calculate θ' for each story, using Equation (10) for all stories above the first and Equation (10*a*) or Equation (10*b*) for the first story.

(4) Calculate R' for each story, using Equation (11*c*).

(5) Calculate $3\,K_C\,R'$ for each column. (For subsequent cycles use $3\,K_C\,R$.)

(6) Separate the joints into two groups of alternate joints—one group to include Joints 1*A*, 1*C*, 1*D*, etc., 2*B*, 2*D*, etc., and the other group to include the remaining joints. For each joint of one group calculate the quantity, $\frac{\Sigma\,(3\,K_C R')}{3\,\Sigma\,K}$, which is the form Equation (12) takes when the points of contraflexure are at the mid-lengths of the members. This is to be considered in the nature of an estimate. For the joints of the first story, instead of the foregoing expression use:

$$\frac{3\,K_{C2}\,R'_2 + \frac{3}{2}\,K_{C1}\,R'_1}{3\,\Sigma K - \frac{3}{2}\,K_{C1}}$$

for columns pin-ended at the foundations; and,

$$\frac{\Sigma\,(3\,K_C\,R')}{3\,\Sigma K - K_{C1}}$$

for columns fully fixed at the foundations. These expressions, easily derived from fundamental slope-deflection considerations, more nearly approximate the true θ-value of a joint under the assumed conditions of fixity.

(7) Using Equation (12), calculate a series of θ-values for the remaining joints, based on the results of Steps (5) and (6).

(8) By the same method as Step (7), calculate a series of θ-values for the first group of joints, based on the results of Steps (5) and (7).

(9) For greater accuracy, the θ-values of Step (7) may be recalculated on the basis of Step (8). Ordinarily, even in the analysis of asymmetrical bents, this step may be omitted as it was in the illustrative problem of Fig. 2.

(10) Using the results of Steps (7) and (8), or of Steps (8) and (9), in Equations (11), (11*a*), or (11*b*), calculate a new set of R-values (in reality this is the first approximation for these R-values, the corresponding R'-values being considered as estimates).

(11) Repeat Steps (5), (7), and (8) (and Step (9) if extreme accuracy is desired) on the basis of Step (10), using for each θ_i in Equation (12) the last value obtained. This gives a new series of θ-values.

(12) Repeat Step (10), using in Equations (11), (11*a*), or (11*b*) the results of Step (11). This gives a new series of R-values.

(13) Repeat Steps (11) and (12) until the desired degree of accuracy is obtained. It will be found that for all but the most irregular cases this step may be omitted. In fact, in many analyses, the desired accuracy will be obtained if the approximations or cycles proceed no farther than Step (10).

(14) Substituting the results of the previous steps in Equations (2) and (3), calculate the end moments of each member.

(15) Calculate, if desired, the horizontal deflection of each story, using Equation (5), and working upward from the first story.

Illustrative Problem

It is desired to find the end moments of the members in the lower part of a 20-story bent as shown in Fig. 2. The wind load, which causes the panel concentrations and shears as shown, is equivalent to a pressure of 30 lb per sq ft on a strip 1 ft wide and extending to the top of the building. All work is to be done on a simple line drawing of the bent (this will be found to be the most convenient method). The columns are to be assumed as fully fixed at the foundations. In order to provide a limit to the problem, it will be assumed that the points of contraflexure of the fourth-story columns are at their mid-lengths; also, it will be assumed that $R_4 = R_5$.

The fourth story is used to show the general form and detail of the calculations. The encircled figures refer to the steps of the foregoing recommended procedure. When considered with this step-by-step procedure, Fig. 2 becomes practically self-explanatory.

Deflection

The deflection at the top of any story is, of course, ΣD up to and including that story. Equation (5) gives the value of any D in terms of its corresponding R and h; that is, $D = CRh$, in which, C is the ratio of the relative stiffness factor to the actual stiffness factor $\left(\text{which is taken as } \frac{2EI}{L}\right)$. Thus, in the problem of Fig. 2, the value of $\frac{I}{L}$ was taken as the K-value for

each member, and:

$$C = \frac{1}{2\,E} = \frac{1}{60\,000\,000}$$

Furthermore, if, as in the illustrative problem, the deflections are desired in inches while the moments are in units of 1 000 lb-ft and the story height is in feet, C must be multiplied by 144 000. The deflection at the top of the first story would be:

$$D = \frac{144\,000}{60\,000\,000}\,(0.458)\,(22) = 0.0242 \text{ in.}$$

Similarly, for the second story,

$$D = \frac{144\,000}{60\,000\,000}\,(0.432)\,(16) = 0.0166 \text{ in.}$$

The total horizontal deflection at the top of the second story is:

$$\Sigma\,D = 0.0242 + 0.0166 = 0.0408 \text{ in.}$$

Since the design load was the wind pressure on a vertical strip, 1 ft wide, extending to the top of the building, true deflections will be given by the product of these values and the distance, in feet, between bents.

Often, perhaps as a preliminary check on the rigidity of the structure, the engineer may desire the value of the primary deflection in advance of having calculated the actual R-values. For this purpose the R'-values, which are easily calculated, may be used.

The Composite Bent

Thus far, a bent has been considered as being the structural unit. In reality, the action of any individual bent may depend quite as much on the action of neighboring bents as it does on the loads directly imposed. In tall buildings, particularly, the floors and walls force all bents to assume approximately the same horizontal deflections. The division of load between the various bents, then, should be made on the basis of some rational conception of unified action throughout the frame.

One of the simplest, yet most rational, methods of analyzing this unified action of a complete frame involves the conception of a composite bent. The general method, as applied to buildings of symmetrical structural layout, includes, briefly, the following procedure: (*a*) Group all bents into one composite bent (the loads, of course, must also be grouped together); (*b*) obtain a fairly accurate series of R-values—in most cases, the initial R'-values will be adequate—for this composite bent; (*c*) using these R-values in the individual bents, calculate a set of consistent θ-values for the various joints; (*d*) calculate a new series of R-values on the basis of the weighted average of the columns of all bents; (*e*) repeat Steps (*c*) and (*d*), if desired; and, (*f*) compute moments.

In the case of special problems, the method is subject to variation. When all parallel bents are exactly the same, a single bent may be analyzed for its proportion of the total load. On the other hand, when the building is completely asymmetrical, some corrective factor may be introduced to take care of the torsion of the frame as a whole.

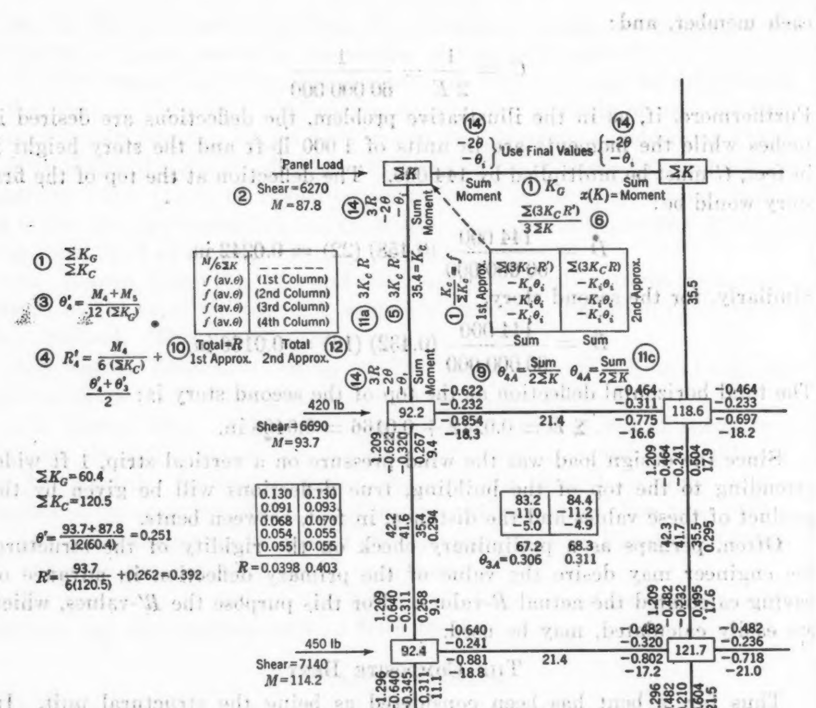
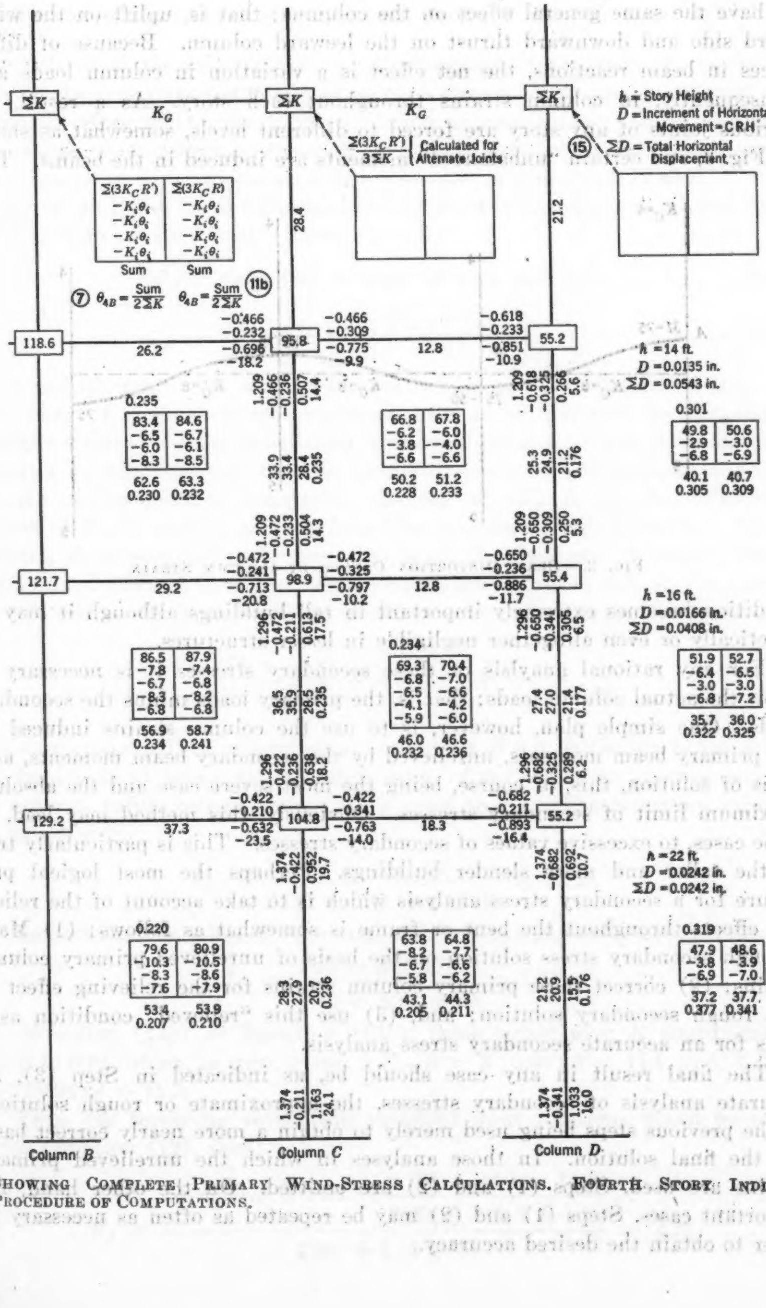


FIG. 2.—ILLUSTRATIVE PROBLEM, LOWER PART OF TWENTY-STORY, ASYMMETRICAL BENT, GENERAL FORM AND STEP-BY-STEP



SHOWING COMPLETE PRIMARY WIND-STRESS CALCULATIONS. FOURTH STORY INDICATES PROCEDURE OF COMPUTATIONS.

Secondary Wind Stresses

As the frame deflects under the action of the wind load, all beams tend to have the same general effect on the columns; that is, uplift on the windward side and downward thrust on the leeward column. Because of differences in beam reactions, the net effect is a variation in column loads and, consequently, in column strains throughout each story. As a result, the various joints of any story are forced to different levels, somewhat as shown in Fig. 3, and certain "unbalanced" moments are induced in the beams. This

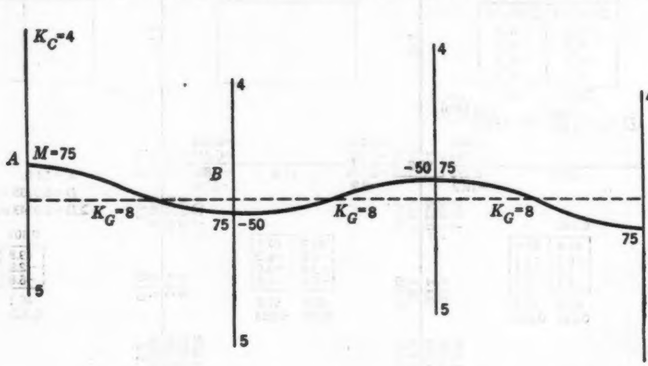


FIG. 3.—BEAM DISTORTION CAUSED BY COLUMN STRAIN.

condition becomes extremely important in tall buildings although it may be practically or even altogether negligible in lesser structures.

For any rational anaylsis of these secondary stresses it is necessary to know the actual column loads; that is, the primary loads minus the secondary loads. One simple plan, however, is to use the column strains induced by the primary beam moments, unrelieved by the secondary beam moments, as a basis of solution, this, of course, being the most severe case and the absolute maximum limit of secondary stresses. Obviously, this method may lead, in some cases, to excessive values of secondary stresses. This is particularly true of the taller and more slender buildings. Perhaps the most logical procedure for a secondary stress analysis which is to take account of the relieving effects throughout the bent or frame is somewhat as follows: (1) Make a rough secondary stress solution on the basis of unrelieved primary column strains; (2) correct these primary column strains for the relieving effect of this rough secondary solution; and, (3) use this "relieved" condition as a basis for an accurate secondary stress analysis.

The final result in any case should be, as indicated in Step (3), an accurate analysis of secondary stresses, the approximate or rough solutions of the previous steps being used merely to obtain a more nearly correct basis for the final solution. In those analyses in which the unrelieved primary strains are used, Steps (1) and (2) are omitted. On the other hand, for important cases, Steps (1) and (2) may be repeated as often as necessary in order to obtain the desired accuracy.

The secondary stress analysis, whether it be a rough preliminary solution or an accurate final solution, must consider the following: (*a*) There exists at each end of each beam an unbalanced fixed-end moment equal to $\frac{6EI}{L}\frac{D}{L}$, in which, D is the relative vertical displacement of the ends of the beam; that is, the cumulative differential column strains up to that beam; (*b*) the joints, therefore, must rotate to new positions of equilibrium; and, (*c*) the various stories must deflect horizontally so that at each story the total shear in the columns, in so far as the secondary stresses are concerned, is zero.

As far as Point (*c*) is concerned, the solution is analogous to a wind stress problem with no wind load. Then,

$$R_n = (\text{weighted average of } \theta_m\text{'s and } \theta_n\text{'s}) \dots\dots\dots\dots (13a)$$

or,

$$R_n = \Sigma\left[f_{nX}\frac{(\theta_{mX}+\theta_{nX})}{2}\right] \dots\dots\dots\dots\dots\dots (13b)$$

As an illustration of a simplified method of analysis, suppose that a certain story of a symmetrical three-span bent is so distorted by cumulative columns strains that the unbalanced moments indicated in Fig. 3 are induced. Bearing in mind the fact that the points of contraflexure are near the mid-lengths of the columns (justifying the use of $3K_c$ in the denominators), values of R, θ_A, and θ_B may be found by successive approximation. If the column, R, is assumed to be zero (the beam R-values are, of course, known and of such amount as to give for the respective values of $3\,K_G R$, the quantities, 75 and −50), Equation (12) would give for a first approximation,

$$\theta_A = \frac{75 - \dfrac{8\,(25)}{2\,(8) + 3\,(4 + 5 + 8)}}{2\,(8) + 3\,(4 + 5)} = 1.68$$

The numerator will be recognized as consisting of $3\,K_G R$ (which is 75) minus K_i (which is 8) times an estimated value of θ_i (which is, in this case, θ_B, all other θ_i values being assumed to be equal to θ_A). The foregoing value of θ_A would give,

$$\theta_B = \frac{25 - 8\,(1.68)}{2\,(8) + 3\,(4 + 5 + 8)} = 0.17$$

Rechecking the value of θ_A, a step which is not generally necessary:

$$\theta_A = \frac{75 - 8\,(0.17)}{2\,(8) + 3\,(4 + 5)} = 1.71$$

By Equation (13*a*) or Equation (13*b*), these would give R approximately equal to 0.94, which, in turn, gives: $3\,K_{cm}R = 3\,(0.94)\,5 = 14.1$, and, $3\,K_{cn}R = 3\,(0.94)\,4 = 11.3$.

From the foregoing values the corrected joint rotations would be found from Equation (12), thus:

$$\theta_A = \frac{75 + 11.3 + 14.1 - \dfrac{8\,(25 + 14.1 + 11.3)}{2\,(8) + 3\,(4 + 5 + 8)}}{2\,(8) + 3\,(4 + 5)} = 2.22$$

and,

$$\theta_B = \frac{25 + 14.1 + 11.3 - 8\,(2.22)}{2\,(8) + 3\,(4 + 5 + 8)} = 0.34$$

Rechecking the value of θ_A:

$$\theta_A = \frac{75 + 14.1 + 11.3 - 8\,(0.34)}{2\,(8) + 3\,(4 + 5)} = 2.27$$

From these values a new R-value may be computed, giving, in this case, approximately 1.30.

From this point forward in the analysis, several variations of method may be used, depending on the accuracy desired:

(1) If the analysis is to be used merely as a preliminary estimate which will furnish a basis for calculating "relieved," or net, column strains, the values of R, θ_A, and θ_B, as found, may be used in Equations (2) and (4) to compute moments. It should be noted that, in such a preliminary solution, it is necessary to calculate only the beam moments.

(2) For the many cases in which the secondary stresses are relatively unimportant, although appreciable, the foregoing values may be used to calculate the moments.

(3) Where a more accurate solution is required, the foregoing method serves as a convenient means of obtaining a series of initial R-values, corresponding to the R'-values of the primary solution, on the basis of which one or more cycles may be applied to obtain final correct values of the secondary displacements. Moments may then be computed from these final values.

On the other hand, a series of cycles may be applied directly to the bent as a whole instead of splitting it up into sections, one of which has been analyzed previously. This straightforward method needs no explanation. Regardless of which method is used, Equation (13a) or Equation (13b) and a modification of Equation (12) which includes $3\,K_G R$, or $\frac{6\,EI}{L}\,\frac{D}{L}$, as well as $3\,K_C R$, will form the basis of solution.

Conclusion

The writer has presented, in the foregoing, a basic method for the analysis of both primary and secondary wind stresses. It must be borne in mind, however, that for some few special cases of the frame as a whole the method is subject to variation, as has been indicated.

This method, it is believed, offers certain definite advantages not generally possessed by other methods now in use, namely: (1) Speed and simplicity are obtained without the sacrifice of accuracy; (2) deflections are obtained directly; (3) the methods require no elaborate instruments; (4) with the conception of the composite bent, the method is easily extended to include the analysis of certain types of complete frames; and, (5) asymmetrical bents are analyzed quite as easily as symmetrical bents.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

REPORTS

EARTHS AND FOUNDATIONS[1]

PROGRESS REPORT OF SPECIAL COMMITTEE

TO THE BOARD OF DIRECTION,
AMERICAN SOCIETY OF CIVIL ENGINEERS:

This Committee was constituted in 1930 and endowed with appropriations from the Society, from Engineering Foundation, and the unexpended funds of the Special Committee on the Bearing Value of Soils for Foundations the work of which it continued in certain ways.

Although much progress has been made in recent years in the field of Soil Mechanics and related subjects, the Committee felt that further research work was necessary, to make practicable the application of this knowledge by engineers.

The research work was pursued at the Massachusetts Institute of Technology, Cambridge, Mass., at Technische Hochschule, Vienna, Austria, at Columbia University, New York, N. Y., and at Yale University, New Haven, Conn.

This Committee believes that this report contains the basic principles by which may be solved the general problem of the settlements of structures erected on soils containing strata of very fine-grained saturated materials.

It is proposed to outline the necessary mechanics including the usable formulas, curves, and tables, with a due appreciation of errors of assumption that may appear in the methods as the theory is modified by later experiment. The basic principles of the mathematics have been developed by Charles Terzaghi, M. Am. Soc. C. E., a member of the Committee, to which principles important contributions have been made by Glennon Gilboy, Jun. Am. Soc. C. E., who succeeded Professor Terzaghi at the Massachusetts Institute of Technology.

Under the direction of Professor Gilboy, with the assistance of S. J. Buchanan, Jun. Am. Soc. C. E., retained for this purpose by the Committee, this Institute has placed the facilities of its Soil Mechanics Laboratory, at the disposal of the Committee. Mr. Buchanan has been employed by the Committee for two years (since 1931) and has rendered valuable service in developing mathematical aids in making more applicable the formulas of soil mechanics as well as in developing a boring machine for obtaining undisturbed samples of clay or similar material.

[1] Presented at the Annual Meeting, New York, N. Y., January 18, 193. Discussion on this report will be closed in September, 1933, *Proceedings*.

The Committee has also been fortunate in obtaining the benefit of Professor Terzaghi's work in Europe, particularly his observations of settlements in existing structures on yielding and unconsolidated strata of soft materials. An interesting experiment on both the distribution of pressures as well as the settlement of a structure on yielding ground has been carried out by Professor Terzaghi on a tank 90 ft in diameter, an account of which is contained in this report.

Realizing that the most important problem confronting engineers is that of long-continued settlements in subsoils in which clay is present, a detailed study of the mechanics of settlement in clay is given and several examples are quoted from actual experience. Of underlying importance is the recognition of the fact that no subsoil study is complete unless a comprehensive picture is made of the entire stress system existing in the upper as well as in the deeper lying strata below the loaded surface, together with the resulting behavior of the material in which these stresses exist. A shallow subsoil study is absolutely misleading. It is necessary, therefore, to develop herein the mechanics of pressure distribution under a given loading, with particular reference to the distribution at great depths below the loaded surface.

The Committee fully appreciates the worth of laboratory analysis in connection with foundation practice and warmly welcomes the co-operation of all such research and experimental stations; but it feels that the essence of foundation study is the collection of settlement data taken from existing structures. It has taken particular pains to gather such data, and makes an appeal to the membership at large to furnish it with records of settlement. To make such data usable a proposed form for such records is presented in this report. Professor Terzaghi has assembled such data from his European, Asiatic, and African experiences. The Committee hopes to be furnished with a steady supply of such authentic data on settlement. With a comparatively small outlay of money the Committee has been able to assemble complete records on an important structure in New England and on numerous structures of varying character in Central Europe and, in addition, has been able to co-operate with research workers in similar fields abroad.

The Basic Problem

A load, applied at the ground surface, distributes itself throughout the ground in accordance with fairly simple laws, the pressures diminishing in intensity both in the vertical and in the horizontal directions away from the axis of the loading. An elemental example of pressure distribution may be used as a preliminary study of the radiation of pressure from a load source. Assume a pile of rollers, Fig. 1, with sufficient frictional resistance between the rollers to prevent lateral motion. The number of rollers in each layer increases in arithmetic progression. A load of unity is applied at the single upper roller, the rollers themselves being weightless. Each roller takes the vertical components to the two rollers in which it is nested. The distribution of vertical components of pressure across a horizontal layer is seen to follow the terms of the binomial expansion, $(\frac{1}{2} + \frac{1}{2})^n$.

A typical pressure distribution is shown in Fig. 1. The maximum value of the pressure in any layer (that nearest the center) is given by,

$$p_z = \frac{1}{2^n} \frac{n!}{\left(\frac{n!}{2}\right)^2} \text{..........................} (1)$$

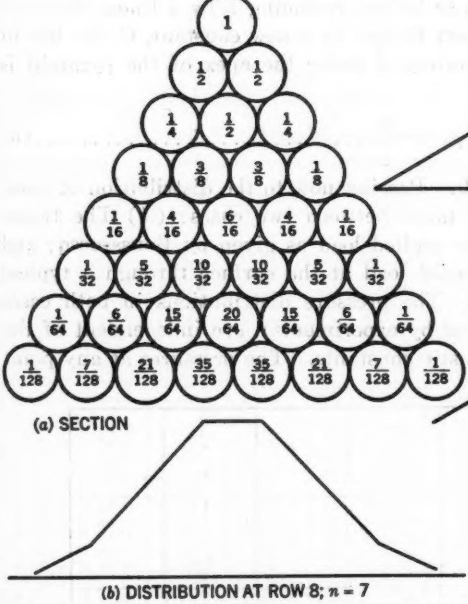


FIG. 1.—PRESSURE DISTRIBUTION AT VARIOUS DEPTHS.

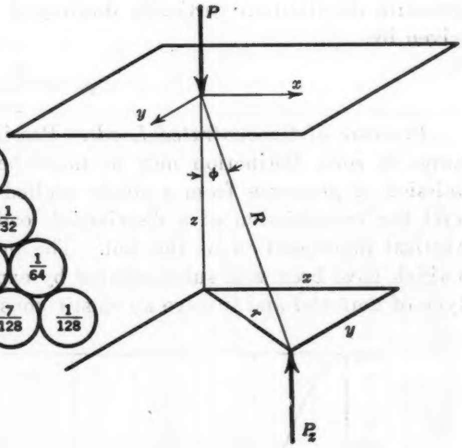


FIG. 2.

in which, $n!$ is the factorial expression, $n\ (n—1)\ (n—2)\ldots.1$. Using an approximate formula for large factorials (Stirling), Equation (1) reduces to,

$$p_z = \sqrt{\frac{2}{\pi n}} \text{..........................} (2)$$

Since the distance, z, vertically downward, is a linear function of the number, n, of the rows, the expression may be written in the form,

$$p_z = \frac{c}{\sqrt{z}} \text{..........................} (3)$$

in which, c is a constant. In similar fashion, taking a four-sided pyramid, each face of which corresponds to the section shown in Fig. 1, comprised of uniform balls with frictional resistance, the number of balls in each layer increases as 1^2, 2^2, 3^2, $\ldots, n^2$. Each ball distributes its load in four equal vertical components to the four in which it is nested. The distribution across a central section at any layer is given by the successive terms in the binomial expansion:

$$p = \frac{1}{2^n} \frac{n!}{\left(\frac{n!}{2}\right)^2} (1+1)^n \text{..........................} (4)$$

and the maximum pressure, that is, the pressure at the axis of the pyramid, is,

$$p_z = \frac{1}{2^{2n}}\left[\frac{n!}{\left(\frac{n!}{2}\right)^2}\right]^2 \quad \text{.......................(5)}$$

Using the same approximation as before, replacing n by a linear function of z, and replacing all other constant factors by a new constant, C, the law of pressure distribution vertically downward under the apex of the pyramid is given by,

$$p_z = \frac{C}{z} \quad \text{.............................(6)}$$

Pressure of Concentrated Loads.—Passing now to the distribution of pressures in soils distinction may be made between two cases: (A) The transmission of pressures from a single applied load as given by Boussinesq; and (B) the transmission of a distributed load at the surface through a typical vertical plane section of the soil. The pressure distributions in both cases (which have been well substantiated by experiments), are independent of the type of material and involve no elastic constants. The pressures at any point

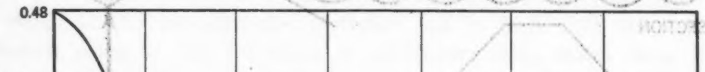


FIG. 3.—VALUES OF K, EQUATION (7)

as given by the theories are stress intensities; that is, they are expressed as vertical load per unit of area. Thus, if the given load is in tons and distance is measured in feet, the pressure as found at any point is given in tons per square foot. Referring to Fig. 2, a load, P, applied at the surface (taken

as the origin of the co-ordinates), develops a stress intensity at the point, r, z, of p_z, or,

$$p_z = K \frac{P}{z^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in which,

$$K = \frac{3}{2\pi} \frac{1}{\left[1 + \left(\frac{r}{z}\right)^2\right]^{\frac{5}{2}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

TABLE 1.—BOUSSINESQ COEFFICIENTS, K, CORRESPONDING TO RATIOS OF $\frac{r}{z}$ (SEE EQUATIONS (7) AND (8))

Ratio, $\frac{r}{z}$	Coefficient, K	Ratio, $\frac{r}{z}$	Coefficient, K	Ratio, $\frac{r}{z}$	Coefficient, K	Ratio, $\frac{r}{z}$	Coefficient, K	Ratio, $\frac{r}{z}$	Coefficient, K	Ratio, $\frac{r}{z}$	Coefficient, K
0.00	0.4775	0.50	0.2733	1.00	0.0844	1.50	0.0251	2.00	0.0085	2.50	0.0034
0.01	0.4773	0.51	0.2679	1.01	0.0823	1.51	0.0245	2.01	0.0084	2.51	0.0033
0.02	0.4770	0.52	0.2625	1.02	0.0803	1.52	0.0240	2.02	0.0082	2.52	0.0033
0.03	0.4764	0.53	0.2571	1.03	0.0783	1.53	0.0234	2.03	0.0081	2.53	0.0032
0.04	0.4756	0.54	0.2518	1.04	0.0764	1.54	0.0229	2.04	0.0079	2.54	0.0032
0.05	0.4745	0.55	0.2466	1.05	0.0744	1.55	0.0224	2.05	0.0078	2.55	0.0031
0.06	0.4732	0.56	0.2414	1.06	0.0727	1.56	0.0219	2.06	0.0076	2.56	0.0031
0.07	0.4717	0.57	0.2363	1.07	0.0709	1.57	0.0214	2.07	0.0075	2.57	0.0030
0.08	0.4699	0.58	0.2313	1.08	0.0691	1.58	0.0209	2.08	0.0073	2.58	0.0030
0.09	0.4679	0.59	0.2263	1.09	0.0674	1.59	0.0204	2.09	0.0072	2.59	0.0029
0.10	0.4657	0.60	0.2214	1.10	0.0658	1.60	0.0200	2.10	0.0070	2.60	0.0029
0.11	0.4633	0.61	0.2165	1.11	0.0641	1.61	0.0195	2.11	0.0069	2.61	0.0028
0.12	0.4607	0.62	0.2117	1.12	0.0626	1.62	0.0191	2.12	0.0068	2.62	0.0028
0.13	0.4579	0.63	0.2070	1.13	0.0610	1.63	0.0187	2.13	0.0066	2.63	0.0027
0.14	0.4548	0.64	0.2024	1.14	0.0595	1.64	0.0183	2.14	0.0065	2.64	0.0027
0.15	0.4516	0.65	0.1978	1.15	0.0581	1.65	0.0179	2.15	0.0064	2.65	0.0026
0.16	0.4482	0.66	0.1934	1.16	0.0567	1.66	0.0175	2.16	0.0063	2.66	0.0026
0.17	0.4446	0.67	0.1889	1.17	0.0553	1.67	0.0171	2.17	0.0062	2.67	0.0025
0.18	0.4409	0.68	0.1846	1.18	0.0539	1.68	0.0167	2.18	0.0060	2.68	0.0025
0.19	0.4370	0.69	0.1804	1.19	0.0526	1.69	0.0163	2.19	0.0059	2.69	0.0025
0.20	0.4329	0.70	0.1762	1.20	0.0513	1.70	0.0160	2.20	0.0058	2.70	0.0024
0.21	0.4286	0.71	0.1721	1.21	0.0501	1.71	0.0157	2.21	0.0057	....	......
0.22	0.4242	0.72	0.1681	1.22	0.0489	1.72	0.0153	2.22	0.0056	2.72	0.0023
0.23	0.4197	0.73	0.1641	1.23	0.0477	1.73	0.0150	2.23	0.0055	....	......
0.24	0.4151	0.74	0.1603	1.24	0.0456	1.74	0.0147	2.24	0.0054	2.74	0.0023
0.25	0.4103	0.75	0.1565	1.25	0.0454	1.75	0.0144	2.25	0.0053	....	......
0.26	0.4054	0.76	0.1527	1.26	0.0443	1.76	0.0141	2.26	0.0052	2.76	0.0022
0.27	0.4004	0.77	0.1491	1.27	0.0433	1.77	0.0138	2.27	0.0051	....	......
0.28	0.3954	0.78	0.1455	1.28	0.0422	1.78	0.0135	2.28	0.0050	2.78	0.0021
0.29	0.3902	0.79	0.1420	1.29	0.0412	1.79	0.0132	2.29	0.0049	....	......
0.30	0.3849	0.80	0.1386	1.30	0.0402	1.80	0.0129	2.30	0.0048	2.80	0.0021
0.31	0.3796	0.81	0.1353	1.31	0.0393	1.81	0.0126	2.31	0.0047	....	......
0.32	0.3742	0.82	0.1320	1.32	0.0384	1.82	0.0124	2.32	0.0047	2.84	0.0019
0.33	0.3687	0.83	0.1288	1.33	0.0374	1.83	0.0121	2.33	0.0046	....	......
0.34	0.3632	0.84	0.1257	1.34	0.0365	1.84	0.0119	2.34	0.0045	2.91	0.0017
0.35	0.3577	0.85	0.1226	1.35	0.0357	1.85	0.0116	2.35	0.0044	....	......
0.36	0.3521	0.86	0.1196	1.36	0.0348	1.86	0.0114	2.36	0.0043	2.99	0.0015
0.37	0.3465	0.87	0.1166	1.37	0.0340	1.87	0.0112	2.37	0.0043	....	......
0.38	0.3408	0.88	0.1138	1.38	0.0332	1.88	0.0109	2.38	0.0042	3.08	0.0013
0.39	0.3351	0.89	0.1110	1.39	0.0324	1.89	0.0107	2.39	0.0041	....	......
0.40	0.3294	0.90	0.1083	1.40	0.0317	1.90	0.0105	2.40	0.0040	3.19	0.0011
0.41	0.3238	0.91	0.1057	1.41	0.0309	1.91	0.0103	2.41	0.0040	....	......
0.42	0.3181	0.92	0.1031	1.42	0.0302	1.92	0.0101	2.42	0.0039	3.31	0.0009
0.43	0.3124	0.93	0.1005	1.43	0.0295	1.93	0.0099	2.43	0.0038	....	......
0.44	0.3068	0.94	0.0981	1.44	0.0288	1.94	0.0097	2.44	0.0038	3.50	0.0007
0.45	0.3011	0.95	0.0956	1.45	0.0282	1.95	0.0095	2.45	0.0037	....	......
0.46	0.2955	0.96	0.0933	1.46	0.0275	1.96	0.0093	2.46	0.0036	3.75	0.0005
0.47	0.2899	0.97	0.0910	1.47	0.0269	1.97	0.0091	2.47	0.0036	....	......
0.48	0.2843	0.98	0.0887	1.48	0.0263	1.98	0.0089	2.48	0.0035	4.13	0.0003
0.49	0.2788	0.99	0.0865	1.49	0.0257	1.99	0.0087	2.49	0.0034	....	......
										4.91	0.0001
										6.15	0.0001

Fig. 3 is a graphic demonstration of the relation between the values of K and the ratio of $\frac{r}{z}$. Table 1 supplies values of the ratio, $\frac{r}{z}$, for computation purposes. The assumption of a load concentrated at a point makes it necessary to go some distance below the point of application of the load to obtain fairly accurate results. As a numerical example, assume a load of 450 tons applied as a point concentration on the surface. Determine the stress intensity at a point 40 ft below the load and 15 ft away horizontally, so that $z = 40$ ft and $r = 15$ ft, and the ratio, $\frac{r}{z} = 0.375$. Entering Table 1 with 0.375, the corresponding value of K is 0.3436. The stress, in tons per square foot, is then $0.3436 \times \frac{450}{40^2} = 0.965$. If a combination of point loads is given, the stress intensities from the several loads may be added at any point to obtain the total pressure at the point from the combination of loads. Thus, Figs. 4 and 5 show the distribution of pressures under three concentrated

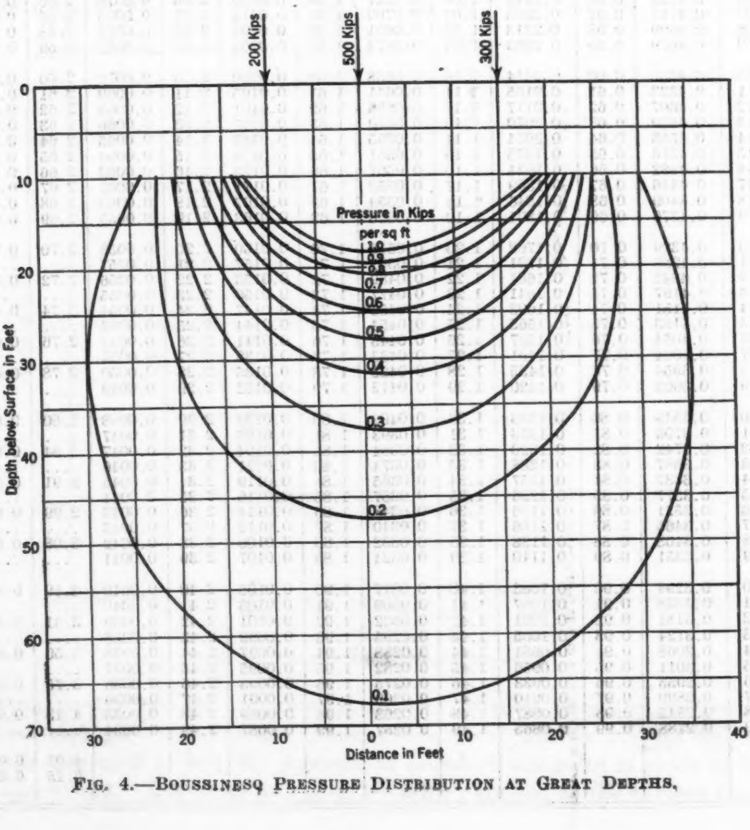


FIG. 4.—BOUSSINESQ PRESSURE DISTRIBUTION AT GREAT DEPTHS.

loads at the surface, as derived from the Boussinesq equations. In order to show more clearly the lines of equal vertical pressure the distribution between the planes from 5 to 15 ft below the load point is shown separately. The curves in Figs. 4 and 5 are the lines of equal vertical intensities. Since

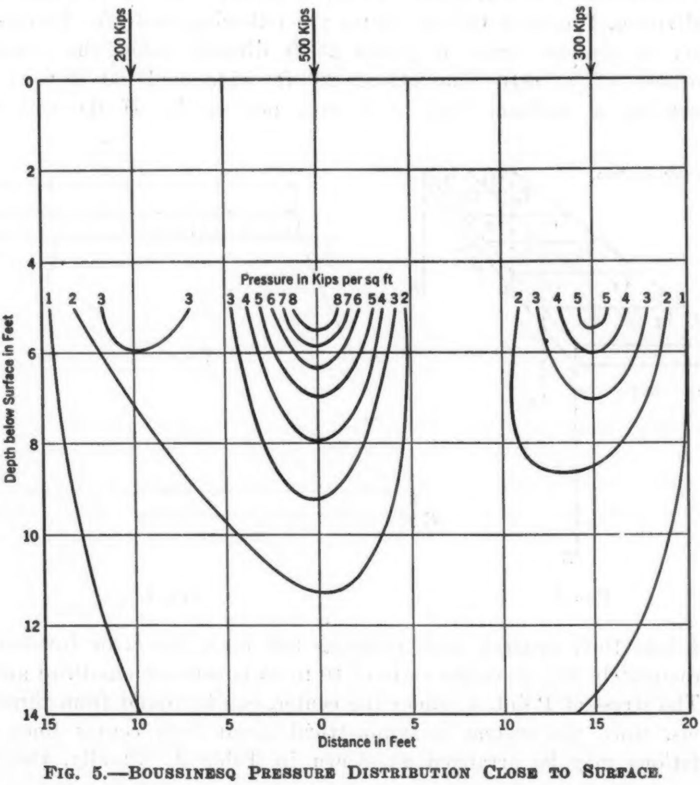


FIG. 5.—BOUSSINESQ PRESSURE DISTRIBUTION CLOSE TO SURFACE.

only plane sections are shown, the actual loci of the equal vertical pressures are bulb-like surfaces. It is noted that a "family" of bulbs forms under each load, the three "families" merging into one bulb at depths of about 15 ft below the surface.

The term, "kips," on the diagrams of this report denotes "kilo-pounds," or 1 000 lb, in each case. It is also to be noted that 1 ton per sq ft is equal, for practical purposes, to 1 kg per sq cm.

In considering the distribution of pressures below a footing of average size the question arises as to the proper subdivision of the footing into units of such size that the load on each unit is sufficiently small to be considered a point concentration as assumed in the Boussinesq formula. Professor Gilboy has made a thorough investigation of this point, and he concludes[2] that "a

[2] Abstracted from Prof. Gilboy's report to the Committee.

division of the area into rectangular elements the longest side of which is less than one-half the distance from the element to the point (at which the stress is to be computed) will give a correct result to within 6%; less than one-third the distance, correct to within 3%; and less than one-fourth the distance, correct to within 2%." As an illustrative example of this process of subdivision, Professor Gilboy quotes the following example: Compute the intensity of vertical stress at points 25 ft directly below the center and the corners of a raft foundation, 20 ft wide and 60 ft long (Fig. 6), carrying a uniform load of 3 tons per sq ft. If the raft is sub-

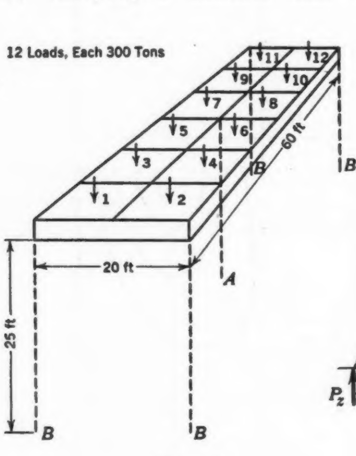


FIG. 6.

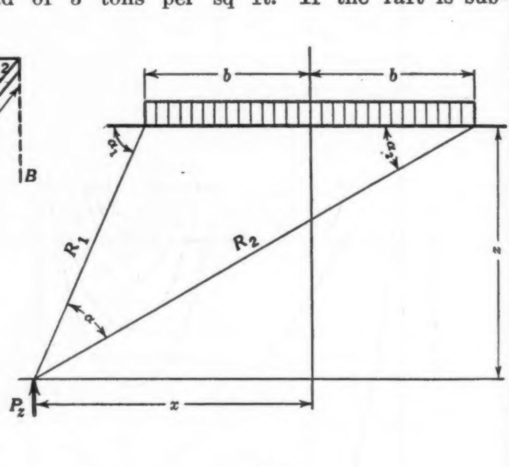


FIG. 7.

divided into 10-ft squares, each carrying 300 tons, the error involved will be approximately 4%, since the ratio of 10 to 25 is between one-third and one-half. The stress at Point *A*, under the center, can be found from three computations, since the system is symmetrical about both center lines. The computations may be arranged as shown in Table 2. Finally, the stress,

$$p_z = 4 \times 0.6802 \times \frac{300}{625} = 1.31 \text{ tons per sq ft.}$$

TABLE 2.—COMPUTATIONS OF VERTICAL STRESS BELOW A RAFT FOUNDATION

Load Nos. (see Fig. 6)	Distance, r, in feet (see Fig. 2)	Ratio, $\frac{r}{z}$	Coefficient, K, in Equation (7)
5, 6, 7, 8	$5\sqrt{2}$	$\frac{\sqrt{2}}{5} = 0.283$	0.3938
3, 4, 9, 10	$5\sqrt{10}$	$\frac{\sqrt{10}}{5} = 0.632$	0.2061
1, 2, 11, 12	$5\sqrt{26}$	$\frac{\sqrt{26}}{5} = 1.020$	0.0803
Total	......	....	0.6802

All the stresses at the corner points, B, in Fig. 6, will be the same. Taking the one nearest Load No. 1 as a basis, the computations may be arranged as shown in Table 3. Finally, the stress, $p_z = 1.1536 \times \frac{300}{625} = 0.55$ tons per sq ft.

TABLE 3.—COMPUTATIONS OF VERTICAL STRESS BELOW A RAFT FOUNDATION

Load No. (see Fig. 6)	Distance, r, in feet (see Fig. 2)	Ratio, $\frac{r}{z}$	Coefficient, K, in Equation (7)	Load No. (see Fig. 6)	Distance r, in feet (see Fig. 2)	Ratio, $\frac{r}{z}$	Coefficient, K, in Equation (7)
1	$5\sqrt{2}$	0.283	0.3938	7	$5\sqrt{50}$	1.414	0.0306
2	$5\sqrt{10}$	0.632	0.2061	8	$5\sqrt{58}$	1.523	0.0238
3	$5\sqrt{10}$	0.632	0.2061	9	$5\sqrt{82}$	1.811	0.0126
4	$5\sqrt{18}$	0.849	0.1229	10	$5\sqrt{90}$	1.887	0.0108
5	$5\sqrt{26}$	1.020	0.0803	11	$5\sqrt{122}$	2.209	0.0057
6	$5\sqrt{34}$	1.166	0.559	12	$5\sqrt{130}$	2.280	0.0050

By proceeding in this fashion with other points, a fairly complete picture of the stress conditions under the area can readily be drawn.

To show in a single section the sum of the intensities along any narrow horizontal strip normal to the section, the total of the stress intensities integrated in the y-direction is given by,

$$s_z = \frac{2\,p}{\pi}\,\frac{1}{z}\,\frac{1}{\left(1 + \frac{x^2}{z^2}\right)^2} \qquad (9)$$

It is noted that for any given value of the ratio, $\frac{x}{z}$, the stress intensity varies inversely as the depth, z, as distinguished from the general case, in which the intensity varies as the inverse square. Equation (9) also gives the intensity at a given point, x, z, in the soil, for a uniformly distributed load along the surface, of p per unit of length.

Pressure Distribution Under Distributed Loads.—Let the load applied at the surface of the ground be p_0 per unit of area distributed over a length of $2\,b$. A plane section of this loading is shown in Fig. 7 and the load at the surface is assumed to extend indefinitely in both directions normal to the plane. The stress intensity at any point, x, z, is given either by,

$$p_z = \frac{p_0}{\pi}\left[\alpha - \sin\alpha\cos(\alpha_1 + \alpha_2)\right] \qquad (10)$$

or,

$$p_z = \frac{p_0}{\pi}\left[\sin^{-1}\frac{2bz}{r_1 r_2} + \frac{2\,b\,z}{r_1^2 r_2^2}\,(z^2 - x^2 + b^2)\right] \qquad (11)$$

in which, $r_1^2 = z^2 + (x - b)^2$; and, $r_2^2 = z^2 + (x + b)^2$.

Fig. 8 gives a typical distribution under a loading of 10 000 lb per sq ft, distributed over a width of 20 ft, that is, $b = 10$ ft. Along the center line of loading (the z-axis), the pressures are given by,

$$p_z = \frac{p_0}{\pi}(\alpha + \sin\alpha) = \frac{p_0}{\pi}\left(\sin^{-1}\frac{2u}{1+u^2} + \frac{2u}{1+u^2}\right) = k\,p_0 \ldots\ldots(12)$$

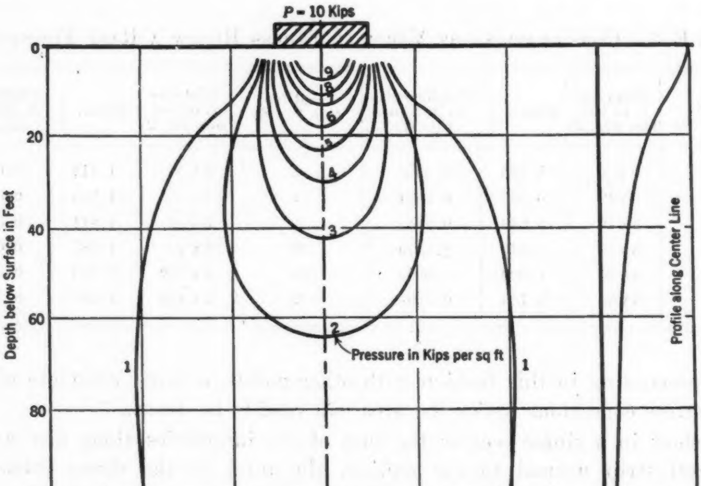


FIG. 8.—PRESSURE CONTOURS; DISTRIBUTED LOADING (1 KIP = 1000 POUNDS).

Table 4 gives the values of k for the ratio, $u = \frac{b}{z}$. Equation (12) may be compared with the pressures found from the Boussinesq formula for a strip loading. Fig. 9 gives the curves for both methods. At depths of about 30 ft (for the given type and amount of loading) the readings become comparable. The method of plane sections as compared with the Boussinesq method has the advantage of giving usable readings closer to the surface.

TABLE 4.—VALUES OF k FOR THE RATIO, $\frac{b}{z}$ $(= u)$

u	k	u	k	u	k
0.0	0.00	0.6	0.63	2.0	0.96
0.1	0.13	0.7	0.69	2.5	0.98
0.2	0.25	0.8	0.74	3.0	0.98
0.3	0.36	0.9	0.80	3.5	0.99
0.4	0.46	1.0	0.82	4.0	0.99
0.5	0.55	1.5	0.92	5.0	1.00

THE MECHANICS OF SETTLEMENT IN CLAY

Clay differs from silt and sand as a foundation material in that settlement takes place through the consolidation of the clay mass as the water is

squeezed out under an applied loading. All clays are assumed to be saturated and to be composed of grains of such minute size that the paths of water flow are extremely restricted. For this reason water escapes with extreme slowness, the rate of flow depending on the permeability of the soil. The grada-

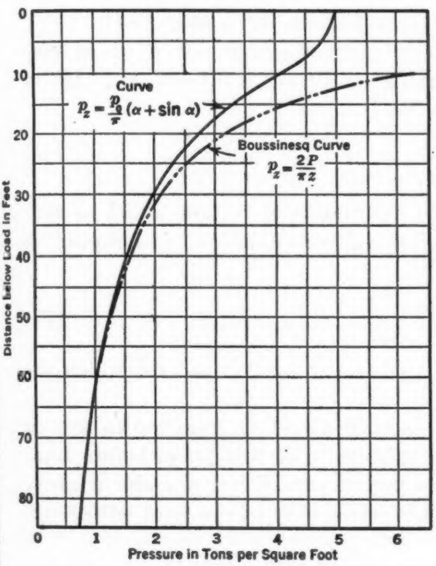


FIG. 9.—COMPARISON OF PRESSURES BY TWO METHODS.

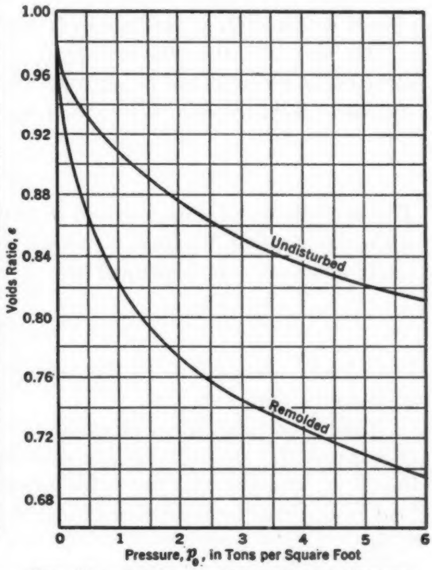


FIG. 10.—PRESSURE, VOIDS-RATIO CURVES.

tions of the different materials, such as gravel, sand, silt, and clay, as well as the laboratory procedure to make such gradations have been set forth in previous publications and need not be discussed. All the tests to determine compressibility, permeability, and other related constants have been well described in several reports and have become fairly well standardized.

The intrinsic properties of a given clay are defined by a permeability coefficient, k, a compression coefficient, a, both experimentally determined, and a derived coefficient, c, the coefficient of consolidation. The given physical state of a clay is defined by a voids-ratio, e, which is the ratio of the volume of voids in a given amount to the volume of the solid content. If n as usual defines the percentage of voids in a given sample, then,

$$e = \frac{n}{1-n} \quad \text{(13)}$$

Thus, if a given sample has 50% voids, $e = 1$.

The compressibility of clay under load is given by a relation between the applied pressure and the voids-ratio. When a load is applied to a prism of clay, at the first instant the water takes all the load. As the water escapes, the load is transferred to the solid skeleton of the prism. When the flow of water ceases under the given load, the voids-ratio is determined for the

clay, for that particular load. A set of laboratory values for a given clay specimen is represented by a smooth logarithmic curve, two of which are illustrated in Fig. 10. Having plotted such a curve for a given clay, values of the voids-ratio for any assignable loading may be taken from this empirical curve. It should be noted that the metric unit pressure, kilograms per square centimeter, is the practical equivalent of tons per square foot.

To simplify the mathematical analysis the thickness of a given clay stratum is reduced to the equivalent thickness of the solid content. If the thickness is h, the equivalent thickness, h_0, is,

$$h_0 = \frac{h}{1 + e} \qquad (14)$$

The coefficients of compression, a, and of permeability, k, are found to vary with the applied loading. The coefficient of consolidation, c, however, is fairly constant. It is defined as,

$$c = \frac{k}{a\,(1 + e)} \qquad (15)$$

in which, e corresponds to the average voids-ratio for the load for which the coefficients were obtained.

Take a vertical prism of small cross-section extending from top to bottom of a given clay deposit. If the average voids-ratio of this prism before loading is e_i, and the average value after complete consolidation under a given load is e_f, then the total shortening of this prism, that is, the total settlement, S, is,

$$S = h\,\frac{e_i - e_f}{1 + e_i} \qquad (16)$$

The total settlement, therefore, depends upon the determination of the initial and final voids-ratio. The distribution of pressure may be found from Equations (7) to (11), and the corresponding voids-ratio taken from curves similar to Fig. 10, proper samples having been taken to evaluate a set of readings for e and p_0.

Rates of Settlement.—Since the total settlement, as given by Equation (16), may not take place for an indefinitely long period, it is important to determine the rate of settlement; that is, the rate at which the materials consolidate. If at any time, t, after the load has been applied, the settlement at a given point is S_t, then,

$$S_t = Q\,S \qquad (17)$$

in which, Q is the percentage of the consolidation and is determined mathematically as follows: Referring to Fig. 11, a clay deposit of reduced thickness, $2\,h_0$, lies between two pervious deposits. (If the lower deposit is impervious, such as rock, take one-half section of the stratum in Fig. 11, or, conversely, take the given reduced thickness to the rock layer and double it). Refer the distribution of pressures to a vertical axis, ZZ. At the instant of loading, that is, for a value of time, $t = 0$, the water content takes all the load (since the material is assumed to be saturated), and the water starts to flow toward

each pervious layer. Each horizontal layer is then under a stress, w, termed the "hydrodynamic excess." The distribution of pressures evaluating w at this time, $t = 0$, is found from the methods outlined previously in this report. Let the line, $B\ A\ C$ (Fig. 11), represent such a distribution of pressure. If

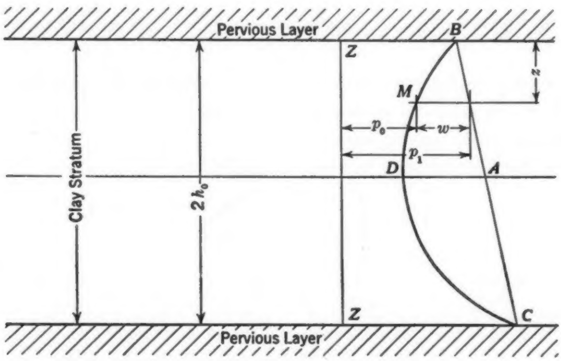


FIG. 11.—CONSOLIDATION OF CLAY.

the distance below the surface is measured by the vertical co-ordinate, z, then the initial pressure distribution may be represented by some function of z, that is, for time, $t = 0$, $w = F(z)$. At the upper and lower surfaces the water escapes at once, so that at these layers the hydrodynamic excesses are zero at all times. Expressed mathematically $w = 0$ for $z = 0$ and $z = 2\ h_0$. At any time, t, the excess pressure curve may be outlined by $BMDC$. At any depth, z, at which the total pressure is p_1, the pressure developed on the solid clay skeleton is given by p_0, and the hydrodynamic excess by w. The rate of escape of the water at this point is a function of the permeability and the compressibility of the soil. A differential equation may be formed by taking a very small prism and noting that the diminution of the volume of this prism must be equal to the quantity of water that escapes through the six faces of this prism. The equation is,

$$c\,\frac{d^2w}{dz^2} = \frac{dw}{dt} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

in which, c is the coefficient of consolidation.[3] A solution may be assumed in the form,

$$w = e^{\beta t + yx} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

in which, e is the exponential base and β is a parameter to be evaluated. Substituting in Equation (19), the only required relation between the parameters is,

$$\beta = c\ y^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

so that one form of solution is,

$$w = e^q\, e^{yz} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

in which, $q = cy^2\, t$.

[3] The derivation of the previous and following formulas is found in Professor Terzaghi's book "Erdbaumechanik."

To introduce trigonometric expressions replace y by iy, in which i is the imaginary expression. This reduces the typical solution to the trigonometric expressions,

$$w = e^{q} \sin yz \quad \text{......................}(22a)$$

and,

$$w = e^{q} \cos yz \quad \text{......................}(22b)$$

These solutions are to be modified so as to introduce the boundary conditions that, $w = w_0$ for $z = 0$; $w = w_1$ for $z = 2h_0$; and $w = f(z)$ for $t = 0$; in which, w_0 and w_1 are constant values, generally zero.

A general solution subject to these conditions is:

$$w = w_0 + (w_1 - w_0)\left[\frac{z}{2h_0} + \frac{2}{\pi}\sum \frac{(-1)^m}{m} e^{-f} \sin \frac{m\,\pi\,z}{2\,h_0}\right]$$

$$+ \frac{1}{h_0}\sum\left(e^{-f} \sin \frac{m\,\pi\,z}{2\,h_0}\int_0^{2h_0}[f(z) - w_0] \sin \frac{m\,\pi\,z}{2\,h_0}\,dz\right) \text{.....}(23)$$

in which, m represents a series of integers starting with 0 and $f = \dfrac{m^2 c\ \pi^2\ t}{4\,h_0^2}$.

Taking w_0 and w_1 as both zero,

$$w = \frac{1}{h_0}\sum e^{-f} \sin \frac{m\,\pi\,z}{2\,h_0}\int_0^{2h_0} f(z) \sin \frac{m\,\pi\,z}{2\,h_0}\,dz \text{.........}(24)$$

With the value of w known, the rate of consolidation, Q, may be found as follows: The average value throughout the layer of the initial hydrodynamic excess, $\dot{w}$, for $t = 0$, is:

$$w = \frac{1}{2h_0}\int_0^{2h_0} f(z)\,dz \text{......................}(25)$$

The average value at any time, t, is,

$$w = \frac{1}{2h_0}\int_0^{2h_0} w\,dz \text{.........................}(26)$$

The value of Q is, therefore,

$$Q = 1 - \frac{\int_0^{2h_0} w\,dz}{\int_0^{2h_0} f(z)\,dz} \text{....................}(27)$$

When a definite functional form has been found for the general expression, $f(z)$, the equations can be reduced to usable forms. The simplest assumption is that the pressure is uniform and constant in value through the depth, z. Term this Case 0. Then,

$$Q = 1 - \frac{8}{\pi^2}\left(e^{-N} + \frac{1}{9}e^{-9N} + \frac{1}{25}e^{-25N}\ldots\right) \text{............}(28)$$

in which,

$$N = \frac{\pi^2\,c\,t}{4\,h_0^2} \quad \text{.........................}(29)$$

Professor Gilboy has extended this analysis to cover the case of a distribution of load that may be represented by a parabolic expression, and in particular has developed a concise expression for the important case of a triangular loading, together with an interpolation formula between this case and that of uniform loading.

For a triangular distribution of pressure, with the same value of N (call this Case 1),

$$Q = 1 - \frac{32}{\pi^3}\left(e^{-N} - \frac{1}{27}\,e^{-9N} + \frac{1}{125}\,e^{-25N} \mp \ldots\right) \ldots\ldots\ldots (30)$$

Table 5 gives the values of Q and N for both Case 0 and Case 1. Let,

$$J = \frac{\log \dfrac{2\,(\pi - 2)\,v + 4}{\pi\,(1 + v)}}{\log \dfrac{\pi}{4}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

If, in a given case, with a trapezoidal distribution such that the ratio of the pressure at $z = 0$ to that at $z = 2h_0$, is v, then the interpolated value of N in Table 5 for a given value of Q is found by taking J times the difference between the two values of N for the given Q from Cases 0 and 1 and adding this interpolated difference to the value of N for Case 0. The values of J for corresponding values of v, are, as follows:

TABLE 5.—VALUES OF Q AND N FOR USE IN SOLVING EQUATIONS (28) AND (30)

Percentage of consolidation, Q	Values of N		Percentage of consolidation, Q	Values of N		Percentage of consolidation, Q	Values of N		Percentage of consolidation, Q	Values of N	
	Case 0	Case 1		Case 0	Case 1		Case 0	Case 1		Case 0	Case 1
0.00	0.00	0.00	0.30	0.17	0.39	0.55	0.59	0.84	0.80	1.40	1.64
0.05	0.005	0.06	0.35	0.24	0.47	0.60	0.71	0.95	0.85	1.69	1.93
0.10	0.02	0.12	0.40	0.31	0.55	0.65	0.84	1.10	0.90	2.09	2.35
0.15	0.04	0.18	0.45	0.39	0.63	0.70	1.00	1.24	0.95	2.80	3.17
0.20	0.08	0.25	0.50	0.49	0.73	0.75	1.18	1.42	1.00	∞	∞
0.25	0.12	0.31	......	....	....	....	....	....	....	....	.....

Value of v	Value of J	Value of v	Value of J
0.0	1.00	0.6	0.27
0.1	0.84	0.7	0.19
0.2	0.69	0.8	0.12
0.3	0.56	0.9	0.06
0.4	0.46	1.0	0.00
0.5	0.36		

Note that the consolidation formulas involve the manner of pressure distribution; not the amount. It is simply necessary to determine the coefficient of consolidation and substitute this value in the expression for N. Generally, the coefficient of consolidation, c, is given in centimeters per minute; the time, t, in years, and the depth, h, in feet. For the units thus described the time, t, in years, corresponding to any value of N, may be found from,

$$t = \frac{N h_0^2}{1\,400\,c} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)$$

Given a clay with a known coefficient of consolidation, c, for any value of N, that is, for any assignable percentage of consolidation, the time to reach that state varies as the square of the thickness of the layer. Thus, assume that a laboratory specimen of a clay layer, 0.02 ft thick, has taken half a day to reach complete consolidation. A layer, 30 ft thick, would require $\left(\frac{30}{0.02}\right)^2$ half days, or approximately 3 100 years to reach complete consolidation.

The consolidation formulas, Equations (28) and (30), may be used to determine the coefficient of consolidation from a laboratory test on a sample. A standardized form of procedure makes the determination of the time to reach 90% consolidation a favorable criterion for an accurate determination of the coefficient. From Table 5, for a uniform load (Case 0), the value of N, corresponding to $Q = 90\%$, is 2.09. Then,

$$c = \frac{0.848}{t} h^2_0 \qquad (33)$$

in which, h_0 is the reduced thickness, in centimeters, and t is the time, in minutes.

The most satisfactory device developed to date for making consolidation tests in clay is that illustrated in Fig. 12. The clay specimen is a cylindrical slice about $2\frac{3}{4}$ in. in diameter and slightly less than $\frac{1}{2}$ in. in thickness, placed in the cylinder (2) between two porous disks, one of which is held in a base (1), the other in a moving piston (5). Both cylinder and base are held by a clamp (3), the piston moving freely within the cylinder. The load is applied through the ball (6), the split plate (7) assuring a concentric applica-

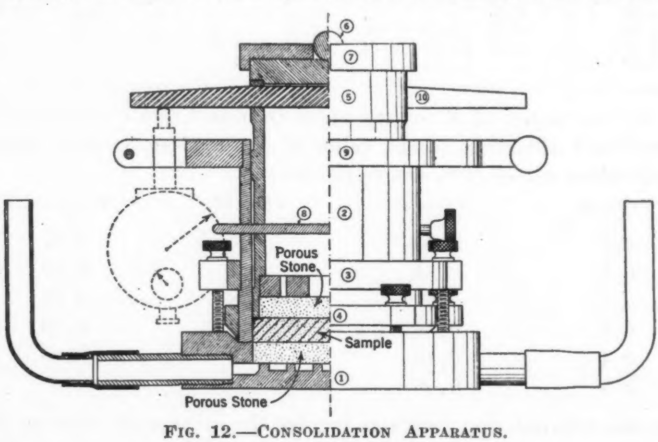


FIG. 12.—CONSOLIDATION APPARATUS.

tion. The amount of shortening or compression is read by two Ames dials (shown dotted) held fast to the cylinder by a ring (9), the stems resting on a bar (10) attached to the piston. The part (4) is a seal ring for rubber packing to make the joint between the cylinder and the base water-tight. While the sample is being placed in the device, the pin (8) holds the piston in the cylinder. The pin is withdrawn when the test begins.

Approximate Pressure Distributions in Deposits not Completely Consolidated.—A load is placed upon a clay layer that has not yet reached complete consolidation prior to the placing of the new load upon it, such as a fill or a river deposit. The approximate distribution of stress for this condition may be noted by referring to Fig. 13. A clay layer of a thickness, $B\ C$, lies below

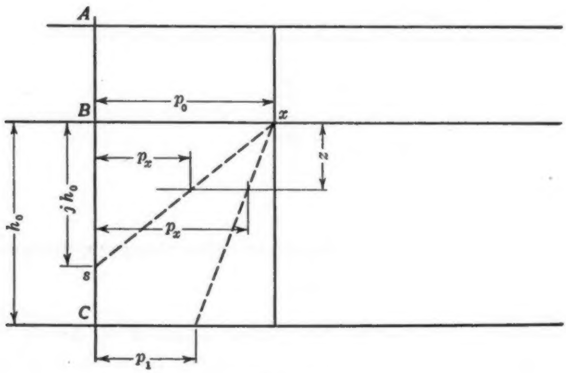


FIG. 13.

a deposit of a thickness, $A\ B$. The line of pressures existing in the solid clay skeleton may be given by the straight line, $x\ s$. Two cases are distinguishable: For Q equal to or greater than one-half,

$$p_1 = p_0\ (2Q - 1) \qquad (34)$$

and, for any value of Q less than one-half,

$$j = 2Q \qquad (35)$$

The pressure distributions as given by Equations (34) and (35) are sufficiently exact for an assumption of original intergranular pressures prior to the application of a load. By intergranular pressure is meant the net pressure acting on the solid clay skeleton; that is, the difference between the actual pressure at the point and the hydrodynamic excess.

Simplified Example of Pressure Distribution and Consolidation.—A simple example will illustrate the principles. A foundation load of 2 000 lb per sq ft (see Fig. 14) over an area indefinite in extent, perpendicular to the section, and 140 ft wide, lies over a silt deposit of fairly recent age, 16 ft thick. Below the silt is a sand layer, quite old, 10 ft thick, and below that is a clay layer, 57 ft thick over rock. It is required to find the progressive settlement at the center of the loading. It is assumed that the clay has reached 100% consolidation under its own load and that of the sand deposit. The silt age being doubtful, it will be assumed that the clay has reached 50% consolidation under the loading produced by the silt. The schematic loading diagrams are given in Fig. 15. The line, $A\ B'$, gives the pressures in clay due to its own weight. The line, $B\ C'$, gives added pressures in the clay due

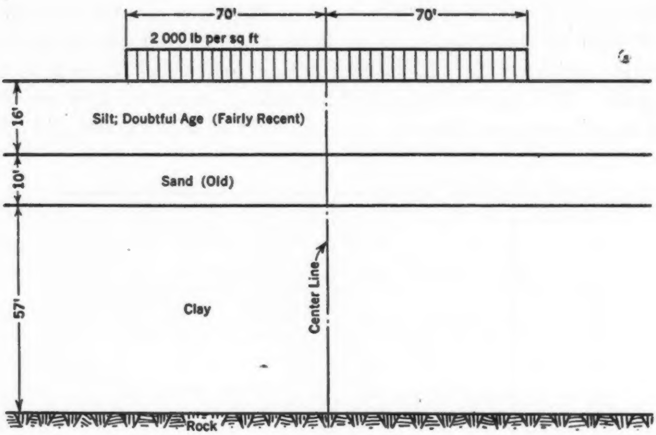


FIG. 14.

to the sand deposit. (All earths have been assumed to weigh 100 lb per cu ft.) The pressure due to the silt deposit on the basis that consolidation has reached 50%, gives a value of the lower pressure of zero (Equation (34)). The line $C\,C'$, is then the pressure line due to the silt. The pressure at any

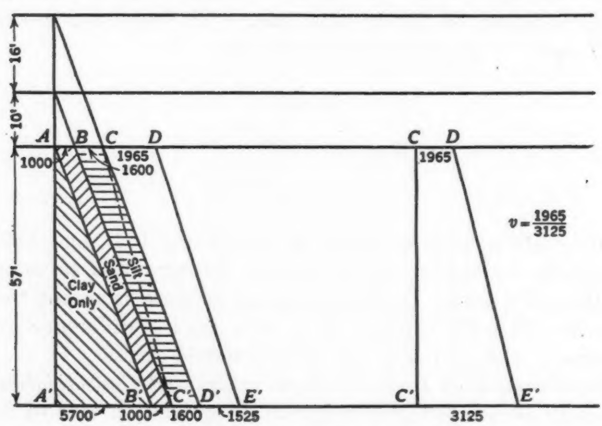


FIG. 15.—LOADING DIAGRAM

depth, z, given by Equation (12), will determine the distribution due to the applied building load. Pressures corresponding to the various depths are, as follows:

Depth, z, in feet	Pressure, p, in pounds per square foot	Depth, z, in feet	Pressure, p, in pounds per square foot
26	1 965	66	1 675
36	1 910	76	1 580
46	1 880	83	1 525
56	1 765	..	.....

The pressure distribution may be represented with reasonable accuracy by a trapezoidal distribution. The pressure line, DE' (Fig. 15), gives this distribution. The actual pressure at the outset of loading ranges, therefore, from 1 965 lb at the top of the layer to 3 125 lb at the bottom. Let it be assumed that the coefficient of consolidation has been found to be 0.0217 cm per min; and that a pressure, voids-ratio series of tests have given the undisturbed curve of Fig. 10. At the outset of loading, the pressure at the top layer is 1.3 tons per sq ft; at the bottom layer, it is 3.35 tons. For the former, the voids-ratio is 0.896, for the latter, 0.850, which gives an average initial voids-ratio of 0.873. After complete consolidation, the top load is 2.28 tons and the bottom load, 4.91 tons, giving corresponding voids-ratios of 0.865 and 0.826, for which the average is 0.849. The total amount of settlement, from Equation (16), is $57 \dfrac{0.873 - 0.849}{1.873} = 0.73 = 9$ in.

The rate of consolidation is then found as follows: At the outset of loading, the top load is 1 965 and the bottom load, 3 125, giving the ratio, $v = \dfrac{1\,965}{3\,125}$, or 0.63. From the list of J-values, $J = 0.25$. The proper values of N for each percentage of consolidation is then one-fourth the interpolated difference between Cases 0 and 1 of Table 4. The reduced value of h is, $\dfrac{57}{1 + \text{initial average voids-ratio}}$; that is, $h_0 = \dfrac{57}{1.873} = 30.4$ ft. With $c = 0.0217$, the time, in years, corresponding to any value of N, is then,

$$t = 30.4\, N \quad (36)$$

Total progressive settlements may then be compiled as shown in Table 6. It is seen that at the end of about 16 years, the settlement should reach 4.5 in. For the sake of simplicity, the zero value of time is taken when the building is one-half completed.

TABLE 6.—Progressive Settlement During Time Required for Complete Consolidation

Percentage of consolidation, Q	Value of N	Time, t, in years	Total settlement, S_t, in inches	Percentage of consolidation, Q	Value of N	Time, t, in years	Total settlement, S_t, in inches
5....	0.02	0.6	0.5	55......	0.65	20	4.95
10....	0.045	1.4	0.9	60......	0.77	23	5.4
15....	0.075	2.3	1.35	65......	0.91	28	5.95
20....	0.12	3.6	1.8	70......	1.06	32	6.3
25....	0.17	5.2	2.25	75......	1.24	38	6.75
30....	0.22	6.7	2.7	80......	1.46	44	7.20
35....	0.30	9.1	3.15	85......	1.75	53	7.65
40....	0.37	11.2	3.6	90......	2.16	66	8.1
45....	0.45	13.7	4.05	95......	2.89	88	8.55
50....	0.55	16.7	4.5	100........	....	....	9.00

The preceding is a skeleton analysis of an actual case of a large building erected upon the deposit of the Charles River Basin near Boston, Mass. Actual settlement levels give maximum settlements, as follows: 1 year, 1 in.; $9\frac{1}{2}$ years, 4.5 in.; and 16 years, 8.25 in., which may be compared with the

theoretical results of 0.75, 3.25, and 4.5 in., respectively, in Table 6. It may be of interest to analyze the actual foundation conditions to ascertain whether, in following a certain procedure to minimize settlement, the settlements were actually augmented. The building was placed upon piles driven into the clay, specified tests showing practical refusal of the pile under loads greater than the final load placed upon the piles. The introduction of piles produced two conditions not assumed in the previous analysis. The mechanics of pressure distribution is different, since the load is no longer placed at the surface, but is taken from the piles in indeterminate amounts by friction. The action of piles in clay is a remoulding effect of uncertain amount, so that the relation between voids-ratio and pressure must be taken from a different curve than that used.

Various conclusions may be made as to the action of the piles. They have a beneficial action inasmuch as they by-pass the upper 15 ft of very compressible silt and, at least, transfer their load to the sand. Below this, their action is detrimental to the clay. A detailed study of this particular case was made by Professor Gilboy and Mr. Buchanan, assuming that the load was transferred to the soil by a uniform frictional resistance and that to the depth penetrated by the piles, the clay was completely remoulded. Modifying the foregoing analysis—using the Boussinesq equations to sum up the pressures from the individual piles, and taking proper voids-ratios from the remoulded as well as from the undisturbed samples (careful borings were taken at locations near the building and at various depths below the top)—a maximum settlement, after complete consolidation, of 29.5 in. is indicated, with a rate of consolidation that follows closely the observed settlements to date. If various pressure contours are drawn at several levels and a relation is formed between the pressure and the settlement at a given period of time, settlement contours may be drawn, which again agree with the observed settlements. It is well to emphasize that since the first approximate analysis indicated a substantial settlement, a variation of a few inches in predicted settlement, depending on the exactness of the analysis, would not have affected the type of design and would have still performed the useful function of indicating the seriousness of the impending movement.

The detailed analysis of the pressure distributions as made by Professor Gilboy and Mr. Buchanan are shown in Fig. 16. The contours of equal pressure distribution are at levels, 25, 55, and 85 ft below the foundation. A vertical section through the pressure contours is also shown. The vertical section shows that at a comparatively small distance below the foundation (25 ft) the surface loading loses its isolated character and blends into the effect of a distributed surface loading. On the basis of these pressure contours, using the local corrective factors noted, a contour of computed and observed settlements has been plotted by Professor Gilboy as shown in Fig. 17.

It is unfair, of course, to criticize the design adopted twenty years ago, in view of the state of foundation science at that time. It is unfortunate,

however, that similar mistakes are taking place in present foundation designs, despite the published data on similar failures. Two procedures may be indicated for a foundation design in this type of material: Excavate the over-

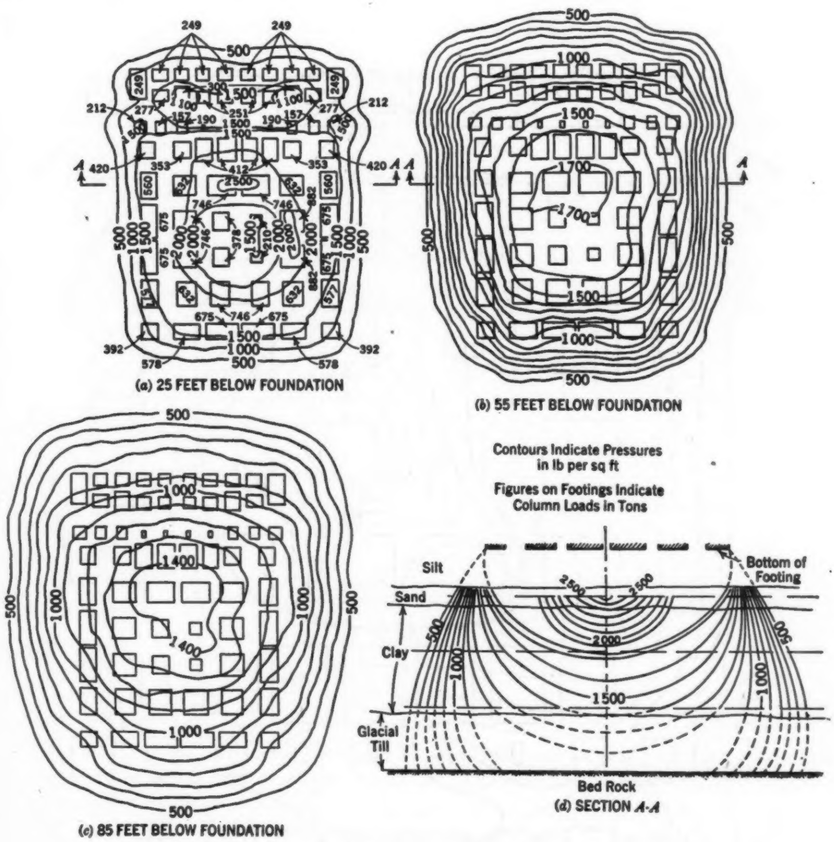


FIG. 16.—DISTRIBUTION OF STRESSES UNDER A TYPICAL BUILDING.

burden of silt, resting the spread footings, or build a stiff truss structure on the sand, using no piles. Settlements would occur, of course, in the amounts indicated in Table 6, but provision could easily be made for jacking the structure back to a level position as the settlement reached stated amounts. The second procedure is to use open caissons, which is a rather costly process, or, better yet, use substantial steel shell piles driven to the material below the clay. The use of such steel piles in similar cases is fast forming an excellent expedient in difficult foundation work.

Analysis of Clay.—As it occurs in Nature, clay is a sedimentary deposit made up of grains of varying shapes and sizes ranging from 0.02 mm down. The formation is controlled by the rapidity of deposit, the nature of the

water (mineral-bearing or fresh), and attendant electrical properties. Several examples of deposits have been found in which the formation is that of linkages of coarser particles. These linkages are well consolidated and transmit all the induced pressures, while within such linkages are masses of unconsoli-

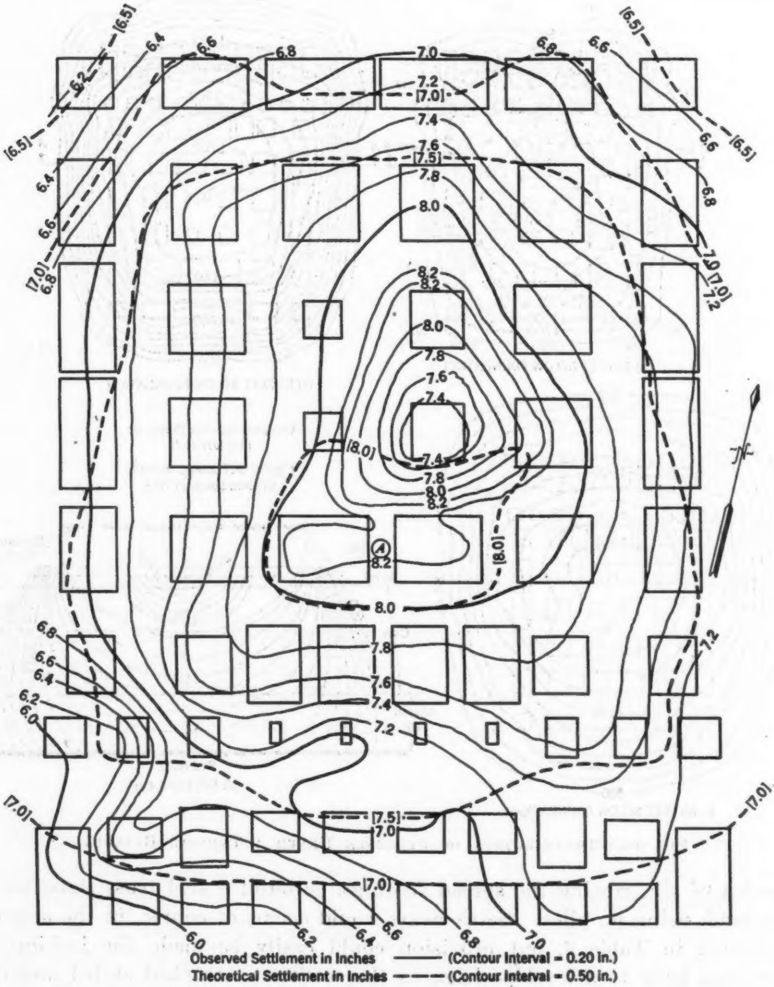


FIG. 17.—CONTOUR MAP SHOWING SETTLEMENT OF TYPICAL BUILDING.

dated clay. When this material is remoulded, either in the laboratory, or by driving piles or sheeting, the deposit becomes an unconsolidated mass with high compressibility. Such a type of clay is exhibited in Fig. 18 by Mr. Arthur Casagrande, to whom is due most of the credit for the investigation of this type of clay. It seems quite probable that all clays possess this linkage formation in some degree so that a remoulded sample is quite different

from a sample taken by borings. An interesting sample of an extreme type of clay is the Laurentian deposit which has been described by Professor Gilboy.[8]

In performing settlement computations, the use of stress-strain diagrams obtained from compression tests on samples moulded to a semi-liquid state

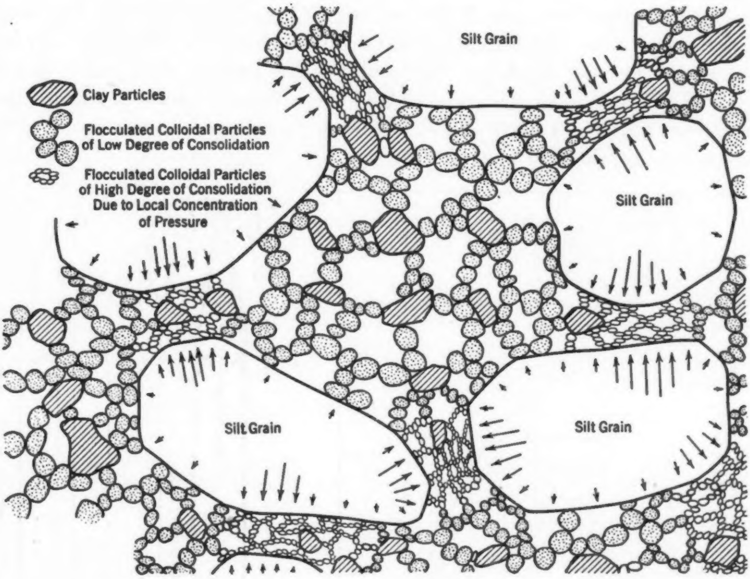


FIG. 18.—STRUCTURE OF UNDISTURBED MARINE CLAY (MODIFICATION 10[4] TIMES).

is not usually justified in the opinion of Professor Gilboy. When such diagrams are used the tacit assumption is made that by starting with the material in a soft state and compressing it, the geological history of the deposit may be reproduced rapidly on a laboratory scale. This assumption seems fairly logical and would be substantially correct were it not for the fact that the properties of the material depend to a considerable degree upon the structure formed in Nature during the process of deposition and subsequent consolidation.

The building up of a clay stratum in Nature is a very slow process. The grains are deposited individually, or in small groups, forming a relatively loose structure. If load were applied suddenly to such a structure, it would tend to cause a decided break-down; because that deposition proceeds very slowly, however, the pressure on any element in the mass increases at an almost imperceptible rate. Consequently, there is a tendency for the original structure to remain substantially intact, and gradually to acquire sufficient resistance to carry the over-burden. If, now, a specimen is removed from the deposit and thoroughly remoulded, so that the structure so carefully built up through centuries is destroyed, it is not to be expected that the

remoulded material will exhibit the same properties as the undisturbed material.

The fact that there is some difference between the properties of undisturbed and remoulded clays has been recognized for a number of years. The great importance of this difference, however, was not realized until quite recently, when Mr. Casagrande had an opportunity to study certain highly consolidated clays from the Valley of the St. Lawrence River. The material in its natural state was almost rock-like in consistency, very hard and brittle, and did not lose its hardness appreciably when immersed in water for many months. When a piece was remoulded in the fingers, however, it changed its consistency quickly to that of soft butter with a water content corresponding about to the liquid limit.

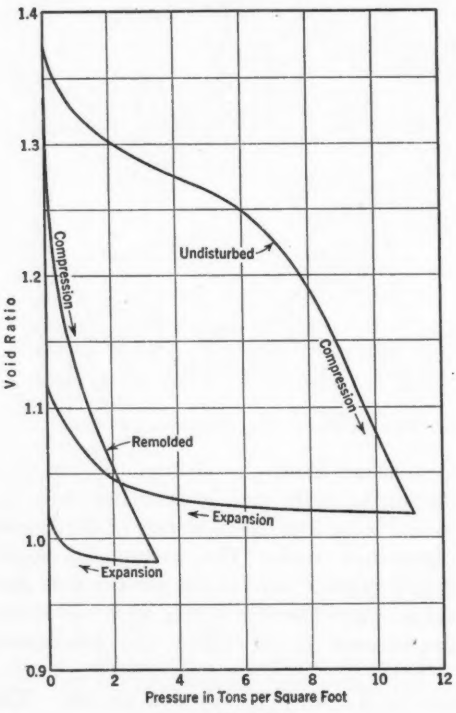


FIG. 19.

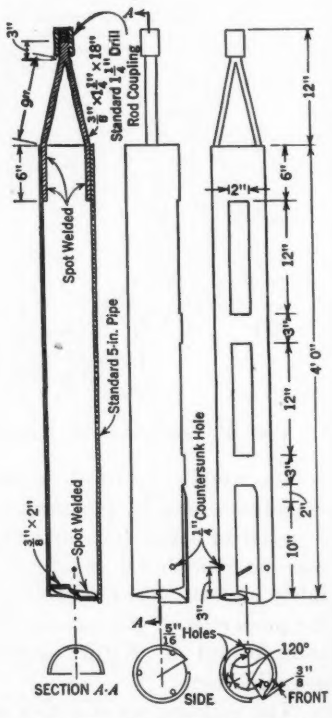


FIG. 20.—FIRST DRILLING TOOL.

This extraordinary behavior required a complete revision of the various ideas previously held concerning the properties of clay. Tests were made on the compressibility of the material in the undisturbed and remoulded states, and here, again, striking differences became apparent. The curves obtained from one pair of such tests are reproduced in Fig. 19. The diagram shows clearly that the break-down of the natural structure increases the compressibility tremendously. The break in the undisturbed curve is interesting because it affords a close idea of the maximum pressure existing in the

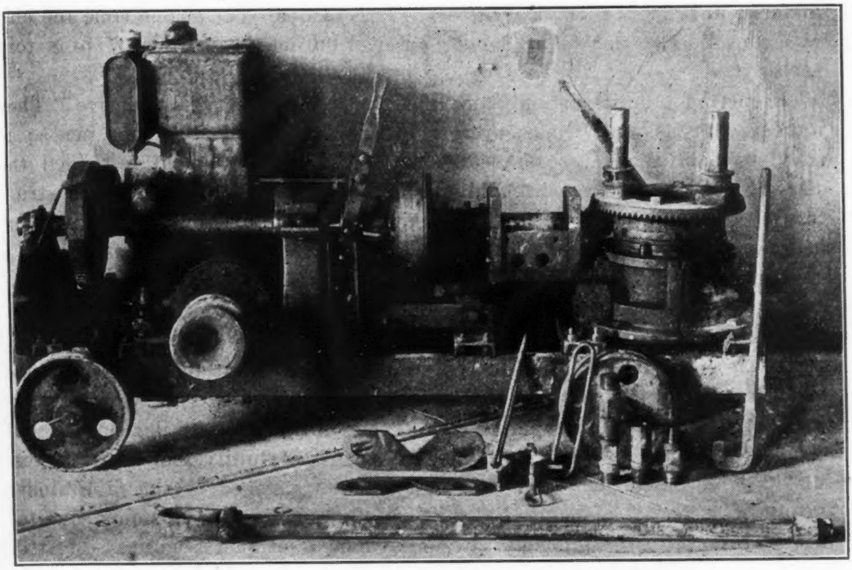


FIG. 21.—VIEW OF BORING AND SAMPLING MACHINE.

FIG. 22.—METHOD OF OPERATING DRILL RODS.

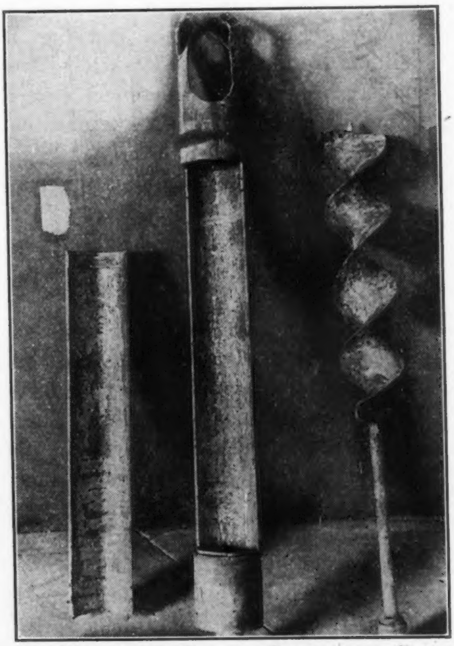


FIG. 23.—AUGER BIT FOR DRILLING IN CLAY.

material during its geological history. It is evident that at one time the clay was consolidated thoroughly under an over-burden pressure of 7 to 8 tons per sq ft.

From the practical standpoint, these curves are very instructive. They indicate, for example, that building loads of as much as 7 tons per sq ft could be applied safely to the clay without much settlement, provided the clay were allowed to remain undisturbed. On the other hand, if the structure of the clay were broken down during construction, by pile-driving, for instance, excessive deformations could be expected under loads as low as 2 tons per sq ft; hence, the rather important inference that, instead of improving the foundations, the driving of piles into clay may make matters much worse.

It was recognized at once, of course, that the Laurentian clay represented an extreme case of structural effect. However, the indications were so striking that it seemed reasonable to expect phenomena of a similar nature, but of a lesser degree, in other clay deposits. An opportunity soon arose to test this idea on Boston clay. A firm of engineers engaged in the preliminary layout of a power plant to be built in the vicinity of Boston requested information as to the probable rate of settlement of the structure. Most of the site was underlaid by a deep bed of the blue clay which is typical of the Boston District. The engineers were asked to obtain samples at least 3 in. in diameter, and as nearly undisturbed as possible, from the clay stratum. In spite of the fact that no unusual precautions in the way of drilling equipment were taken to insure that the samples would be truly undisturbed, marked differences were observed between the compressibility of the relatively undisturbed clay and that of the same sample remoulded.

In short, the conclusion was reached that no adequate concept of the behavior of clay as a foundation material could be obtained without some investigation of the properties of the clay in its undisturbed condition. Accordingly, attention was directed to the development of new methods of boring, sampling, and testing which would serve to make such investigations possible.

Methods of Sampling.—As tests showed the great difference between the properties of undisturbed clay and the same material remoulded, puddled, or disturbed, it became necessary to devise a machine that would not only bore through the various materials at the type localities, but could also recover undisturbed samples for tests. As the compression machine used samples 3.25 in. in diameter, this dimension controlled the drilling and sampling equipment, and standard piping of requisite size was adopted. Where necessary, 6-in. casing was used, which readily permitted the passage of sampling tools, augers, etc., 5 in. in diameter, or less, which were used in the work.

The first boring and sampling machine was a small standard rotary drill, such as those used for shot drill work. This operated a $1\frac{1}{4}$-in. drill rod with the usual couplings. These couplings, in turn, were attached to a drilling tool composed of a 4-ft section of 5-in. pipe arranged as in Fig. 20. Later,

the machine was replaced by a more powerful one illustrated in Fig. 21. The rotary (8 in. in diameter) is operated by a 6-hp gasoline engine geared to operate at 60 rpm. The elevating tackle is operated by an auxiliary winch. The chassis is 6 in. by 3 ft. and the weight of the machine is 1 500 lb. A derrick 15 ft high was used, although a higher frame would have expedited the work greatly. The method of operating the drill rods and

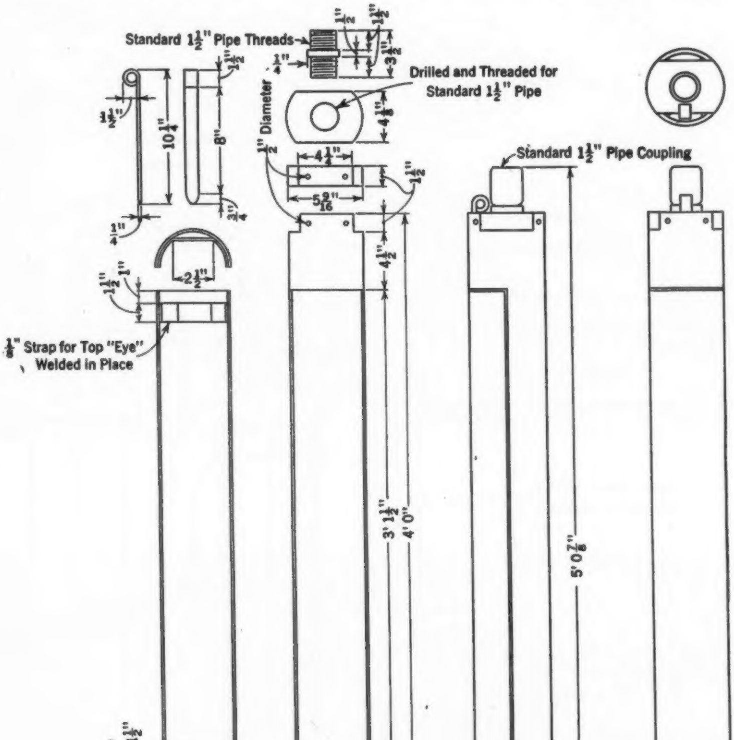


FIG. 24.—SECOND DRILLING TOOL.

allowing them to feed into the hole, is illustrated in Fig. 22. The square top section of the drill rod, known as the "grief stem" is about 1½ in. square. The plates rotating the rods are ¾ in. by 4 in., each with two holes, 2 in. in diameter, 10 in., center to center, and each with a right-angled notch

to fit the "grief stem." By this device the rods are related and readily disconnected. The hollow rotary head allows the quick insertion of the drilling and sampling tools. The grief stem is a square rod 5 ft long, with swivel on top, as shown, and a standard drill rod and thread at the lower end. The drill rods were 10-ft sections of $1\frac{1}{2}$-in. extra strong pipe, with extra strong

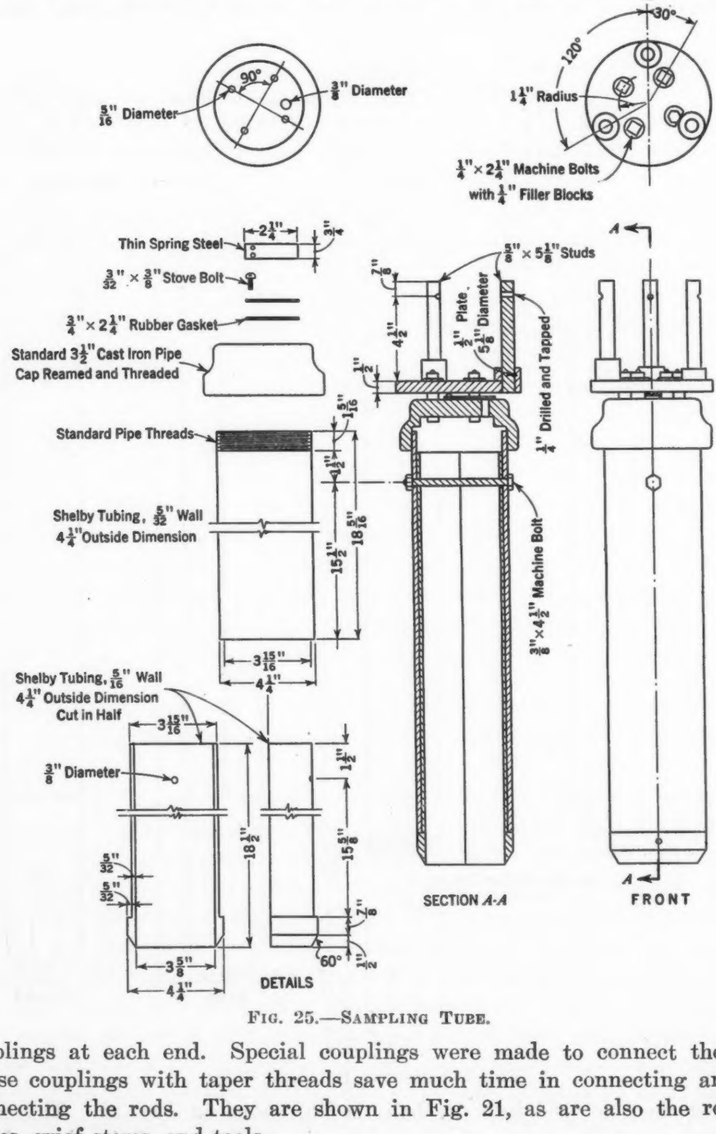


FIG. 25.—SAMPLING TUBE.

couplings at each end. Special couplings were made to connect the rods. These couplings with taper threads save much time in connecting and disconnecting the rods. They are shown in Fig. 21, as are also the rotating plates, grief stems, and tools.

A new drilling tool was designed, as shown in Fig. 23. This tool (see, also, Fig. 24) proved to be useful for soils, such as sands and non-cohesive

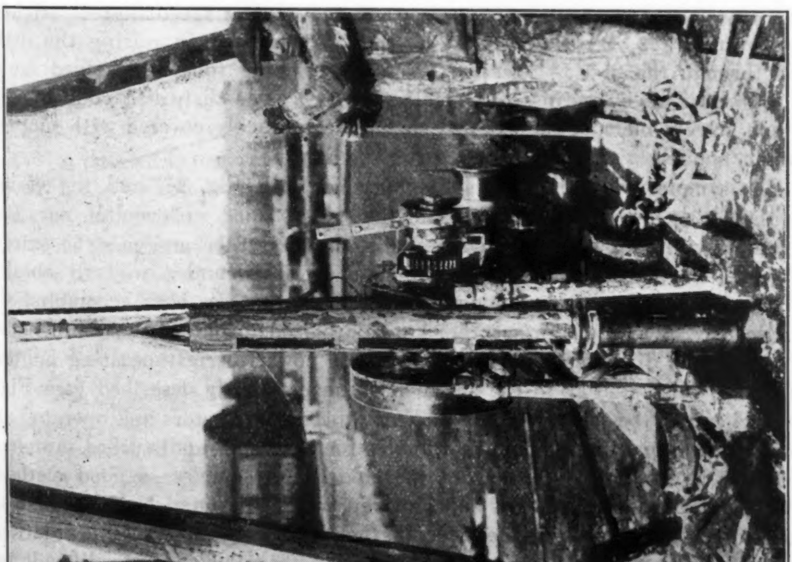


FIG. 26.—DETAILS OF SAMPLING TUBE AND METHOD OF RAISING.

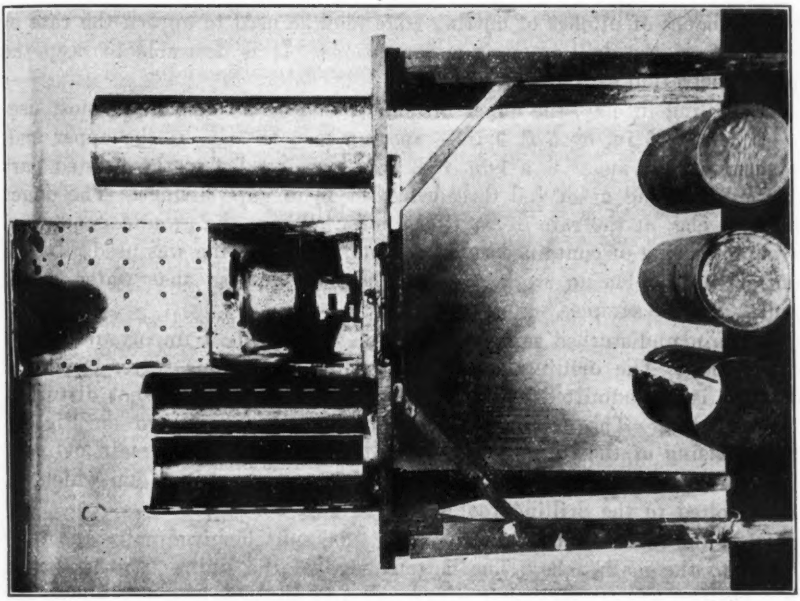


FIG. 27.—SPLIT TUBE FOR CLAY SAMPLES.

materials. The cutting element of 5-in. pipe, 12 in. long, is removable, and different shapes of pitches of cutting edge may be used to govern the rate of penetration of the drill in various formations. It is desirable to keep the rate of penetration as high as possible.

For drilling in clay, the auger bit shown in Fig. 23 was found most useful. It is 2 ft 6 in. by 5 ft 9 in.—tapering to 5 ft 7 in. at the upper end. The shank of the auger is a 1-in. rod, 2 ft long, welded to the twisted part with the upper end upset and threaded to fit $1\frac{1}{2}$-in. pipe fittings. The auger penetrated clay at the rate of $2\frac{1}{2}$ ft per min. Where the ground penetrated was water-bearing or contained running sand, a 6-in. casing was used. It was essential in this boring work to keep the hole dry, so that water would not mix with the samples.

To obtain undisturbed samples of the clay is the most important part of the operation. The drilling can be performed in various ways, providing that water is not admitted to the hole, or that the material is not disturbed before sampling. The details of the sampling tube are shown on Fig. 25 and the raising of the tube in Fig. 26. It is in two parts, a main cylinder $4\frac{1}{4}$ in. in diameter and $18\frac{5}{16}$ in. long, screwed into a cast-iron cup which, in turn, is bolted to the drilling tool shown in Fig. 26.

Within the main cylinder is a smaller tube split longitudinally and telescoped into the main tube. The bottom edge of the lining tube is shaped with a cutting edge. Note that a flap valve in the cast-iron cap allows the escape of entrained air during the sampling and prevents inflow of air later.

The entire sampling tube is lowered into the hole and driven into place by a 125-lb drive weight at the top of the rigid drill rods. The driving is carefully done and the sample sheared off by twisting the drill rods with a pipe wrench. A very slow and steady pull is necessary in raising the drill rods through the first few feet. The inner sampling tube is detached from the outer barrel by means of a small windlass. The disturbed ends of the sample are cut off, and the exposed surfaces are quickly covered with molten paraffin applied with an ordinary soft paint brush.

The sample is placed in a split tube shown in Fig. 27, and the $\frac{3}{16}$-in. annular space between the sample and the tube is filled with molten paraffin. A sample will be preserved indefintely in the split tube and may be safely shipped. At present (1933), experimental work is under way to obtain samples 5 in. in diameter; but nothing definite as yet has been accomplished.

Recently, several manufacturers have developed machines that bore holes several feet in diameter. These machines are powerful rigs operating augers and tools, quite similar to the small machine previously described (see Fig. 21). They may be self-propelling and mounted upon tractors and operated as cranes. Such machines should be valuable for obtaining undisturbed samples. Men may be sent to the bottom of the hole to cut samples—a good method being by the use of thin brass tubes driven into the ground by hand mauls. These tubes may be cut in desired lengths and placed in the consolidation machines. In many cases where borings are not available or are difficult to operate, shafts may be sunk economically by hand, using a small winch

to operate a bucket. The bucket may be raised and lowered by a rope leading to a pulley or over-head portable tripod. Horizontal wooden sheeting may be used to penetrate sand, and vertical sheeting, with iron staves, for clay, as in the familiar Chicago well method. Samples may be gathered at the bottom of the hole sunk by hand. The rig necessary to sink pits by the Chicago well method is not costly, and shafts may be sunk for a few dollars per foot. The advantage of sinking an open shaft is that thorough inspection of the material may be made *in situ,* and samples obtained at any depth and in large numbers. In this connection, it is well to emphasize the fact that several holes are necessary to give reliable results for a large area.

Reports of Actual Settlement

The Committee recognizes that it is of the utmost importance to have the theory of settlements, even if based on the soundest mathematics and laboratory tests, checked as against observed settlements of actual buildings. For this purpose it is necessary to have long-time observations, together with reliable information as to the underground and the loadings imposed. It is surprisingly difficult to obtain reliable settlement data of important structures. There is a strange reluctance on the part of engineers and owners to release such information for publication even when obtainable.

Standard Procedure for Recording Observations.—The general public seems to be under the impression that any engineer ought to be able to construct a foundation so as to insure it against settlement, and is unaware of the yet imperfect state of the science of foundations. In this respect there is perhaps a large proportion of the Engineering Profession in the same state, still unaware that the old simple rules-of-thumb—so many tons per square foot; so many tons per pile—have been proved over and over again to be inadequate and in some cases disastrous. With this in mind Professor Terzaghi has gathered for the Committee records of several notable cases in Europe which are presented herein, including a few cases in the United States. In several notable cases, the Committee regrets that it is not authorized to publish information in its hands, or is not able to give the exact location of the structures in question. Professor Terzaghi has devised a compact method of showing the necessary information in connection with structures.

The Committee feels that complete records of settlement of existing structures is the most vital factor in foundation analysis and, therefore, it has recommended for adoption the method of keeping such records as suggested by Professor Terzaghi. Data on foundation behavior should be presented in the following manner:

(*A*) A plan should be prepared showing the location of the building or structure, with borings and bench-marks. The results of test borings should be presented in graphical form.

(*B*) A simplified plan should be drawn of the foundation of the building (all details omitted) showing: (1) Loads resting upon individual footings or on a mat; (2) soil pressures per unit of area for the different parts of the foundation; and (3) location of bench-marks and reference points. Dead loads, actual live load, and maximum live load, as specified by the local code,

should be indicated separately. Typical sections of the foundation may be added (with a minimum of details), including the geological cross-section of the soil as obtained from borings and other records.

(*C*) Results of soil tests, loading tests, etc., if any were taken.

(*D*) Time-settlement curves for the reference points and also time-load diagrams for the same period. If possible, curves of equal settlement for various stages of the subsidence, should be drawn.

While the Committee has data on a number of cases of settlement, only a small proportion possesses the completeness of information outlined in the foregoing schedule, and, for that reason, only a small part of it is worthy of publication. Data on settlement, of course, are very difficult to obtain. Due to the erroneous assumption that a well-designed foundation does not settle, any observed settlement has been viewed as a partial failure, so that neither the owner, the contractor, nor the engineer cares to publish the data.

The Committee publishes the following cases of settlement as conveying more than the usual interest in such phenomena and as illustrative of characteristic behavior in the consolidation of clay.

Case A.—A Factory Building.—The dimensions of the building are approximately 500 by 460 ft, carried on isolated piers resting on piles varying in length from 65 to 75 ft. The surface load varied from 1.5 to 2.5 tons per sq ft. Fig. 28(*a*) is the geological profile that indicates a layer of hard

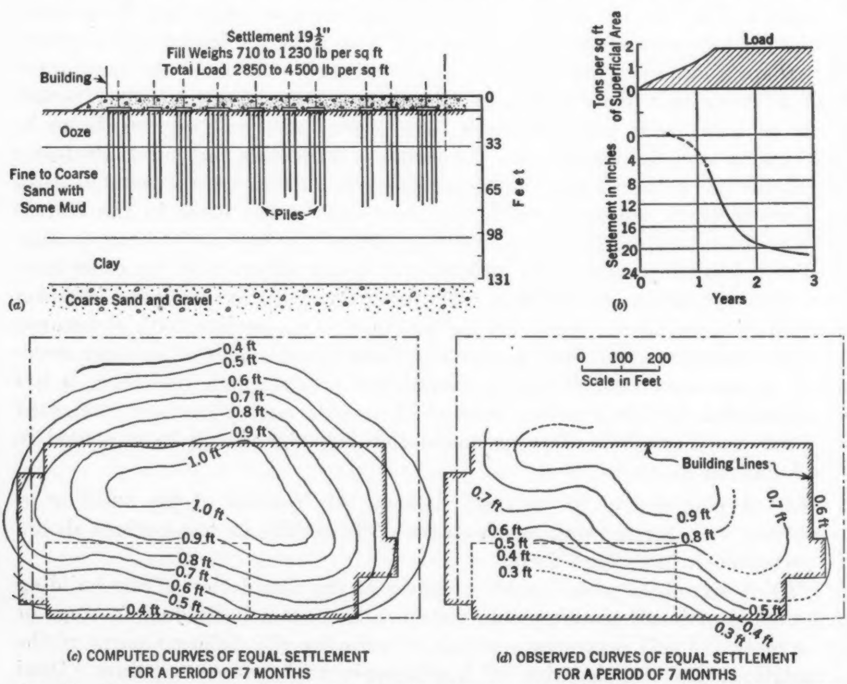


FIG. 28.—LOADING AND SETTLEMENT OF A PILE FOUNDATION (SETTLEMENT DUE TO GRADUAL CONSOLIDATION OF LAYER OF CLAY).

material at a depth of about 130 ft. Overlying this was a bed of stiff clay about 30 ft thick, with about 60 ft of a mixture of fine to coarse sand interspersed with layers of mud. The piles were driven into this layer. Above this thick stratum were layers of silt and recent fill. As usual, the piles were driven to satisfactory penetration, but settlement began at once.

Professor Terzaghi made an analysis of the foundation as outlined herein, determining the load distributions and the rates of consolidation at various points. The curves showing time and load and time and settlement are given in Fig. 28(*b*). The settlement at the end of two years has reached 20 in. The computed and observed settlement contours are shown in Figs. 28(*c*) and 28(*d*), respectively. The agreement is again sufficiently close to confirm the assumptions made in the analysis and the uselessness of the piles as driven.

Case B.—Oil Tanks and Pumping Plant.—A group of five tanks and a pumping plant, placed unsymmetrically in a building adjacent to a tidal stream in the East Indies rested on spread footings. The soil profile is shown in Fig. 29(*b*). The observed settlements on one of the tanks was

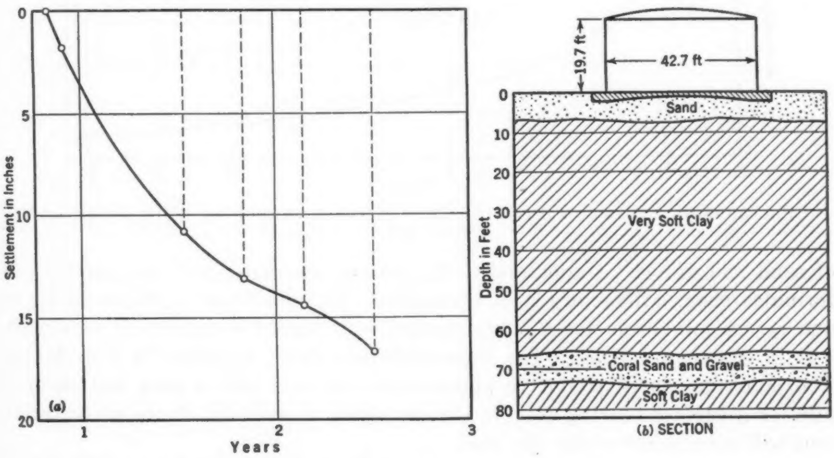


FIG. 29.—SETTLEMENT OF TANK ON THICK LAYER OF SOFT CLAY, BELAWAN, JAVA.

kept with great accuracy. The settlement is substantial, but was anticipated, provisions having been made in both the machinery and tanks for such movement. It had been noticed, in this vicinity, that pile foundations showed great settlement and, in the interests of economy, mat foundations were adopted. Loading tests on the upper sand layer would have been misleading.

Case C.—Post-Office in Austria.—The records here are unusually complete, covering a period of almost twenty-years. A layer of clay about 15 ft thick lies between two layers of permeable sand, and consolidation has caused a settlement of almost 2 ft. The profile is shown in Fig. 30(*a*), and the time-load and time-settlement curves in Fig. 30(*c*). Piles driven into the sand layer may have proved efficacious in this case.

Case D.—The foundation illustrated in Fig. 31 is one of frequent occurrence in engineering practice and, therefore, dangerous because of the feeling of safety engendered by the deep penetration of the piles into a layer

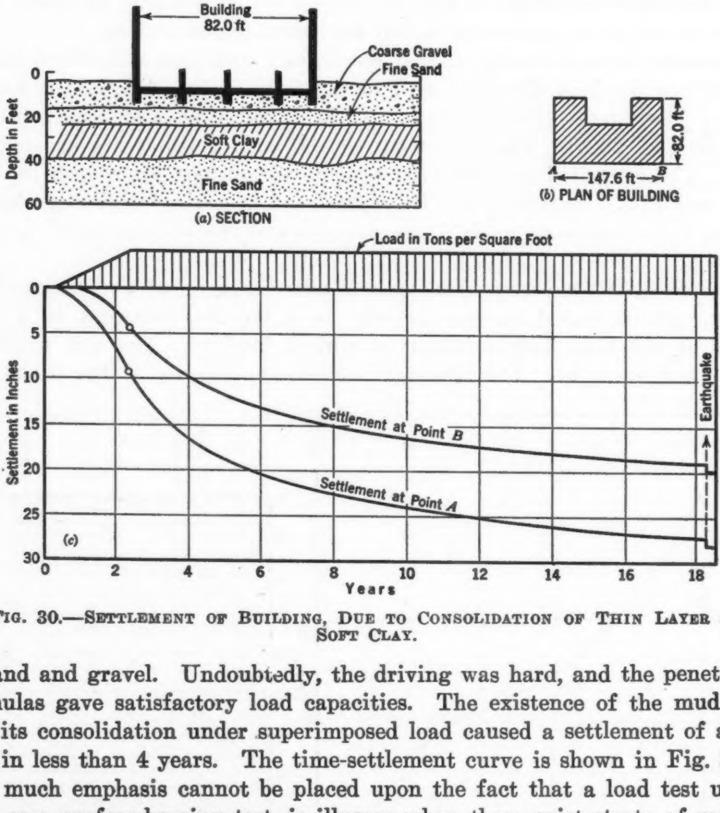


FIG. 30.—SETTLEMENT OF BUILDING, DUE TO CONSOLIDATION OF THIN LAYER OF SOFT CLAY.

of sand and gravel. Undoubtedly, the driving was hard, and the penetration formulas gave satisfactory load capacities. The existence of the mud layer and its consolidation under superimposed load caused a settlement of almost 1 ft in less than 4 years. The time-settlement curve is shown in Fig. 31(*b*). Too much emphasis cannot be placed upon the fact that a load test upon a pile, or a surface-bearing test, is illusory when there exist strata of material that will consolidate under the load.

Case E.—The National Theatre in the City of Mexico, Mexico (see Fig. 32), is one of many old public buildings that have experienced extraordinary settlements (as much as 5 ft), due to the slow consolidation of the volcanic ash which comprises the crater filling in which the city is founded. Only the ability to experience a large amount of plastic flow prevented the complete destruction of many of these fine buildings. Obviously, piles would have added nothing to the stability of the foundation (the layer of volcanic ash is several hundred feet thick and saturated), and the mat footing was more effective than the pile footing. Although all the footings were designed to give the same load intensity on the foundation, the dished effect[4] of the settlement confirms the actual pressure contours as analyzed in the first part of the report.

[4] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 337, Fig. 30.

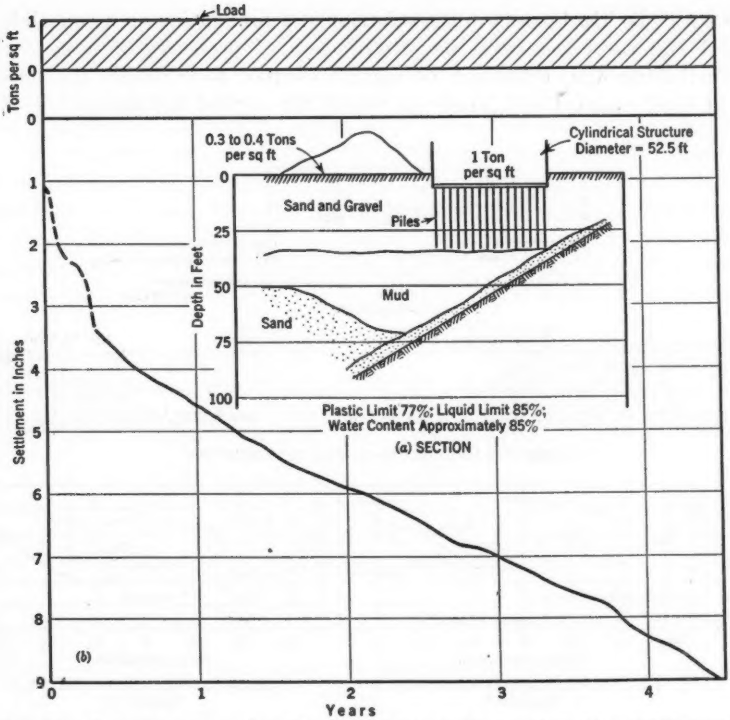


FIG. 31.—SETTLEMENT OF OIL TANK ON WOODEN PILES, DUE TO CONSOLIDATION OF MUD LAYER.

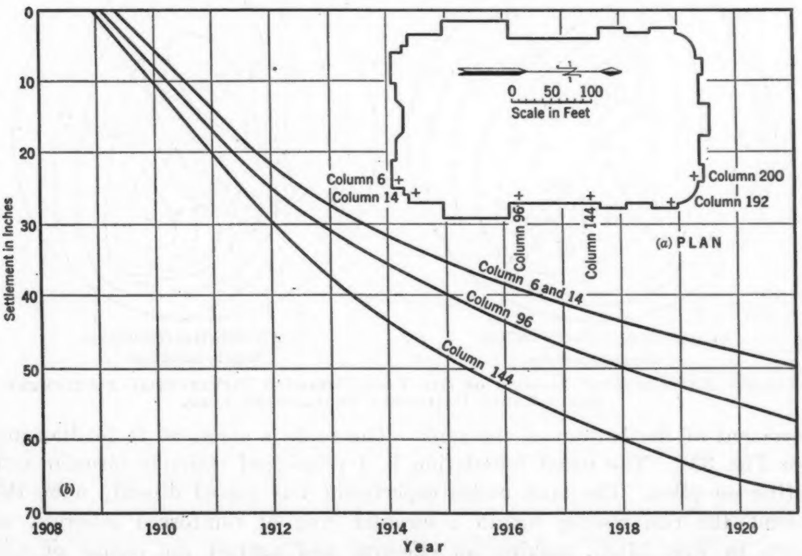


FIG. 32.—SETTLEMENT CURVES OF VARIOUS COLUMNS, NATIONAL THEATRE, CITY OF MEXICO, MEXICO.

Case F.—To illustrate both the behavior of a foundation on a consolidating material and the effect upon the distribution of pressures of a very yielding distributing mat, Professor Terzaghi undertook to re-design the standard form of tank support and made provision for careful determination of the

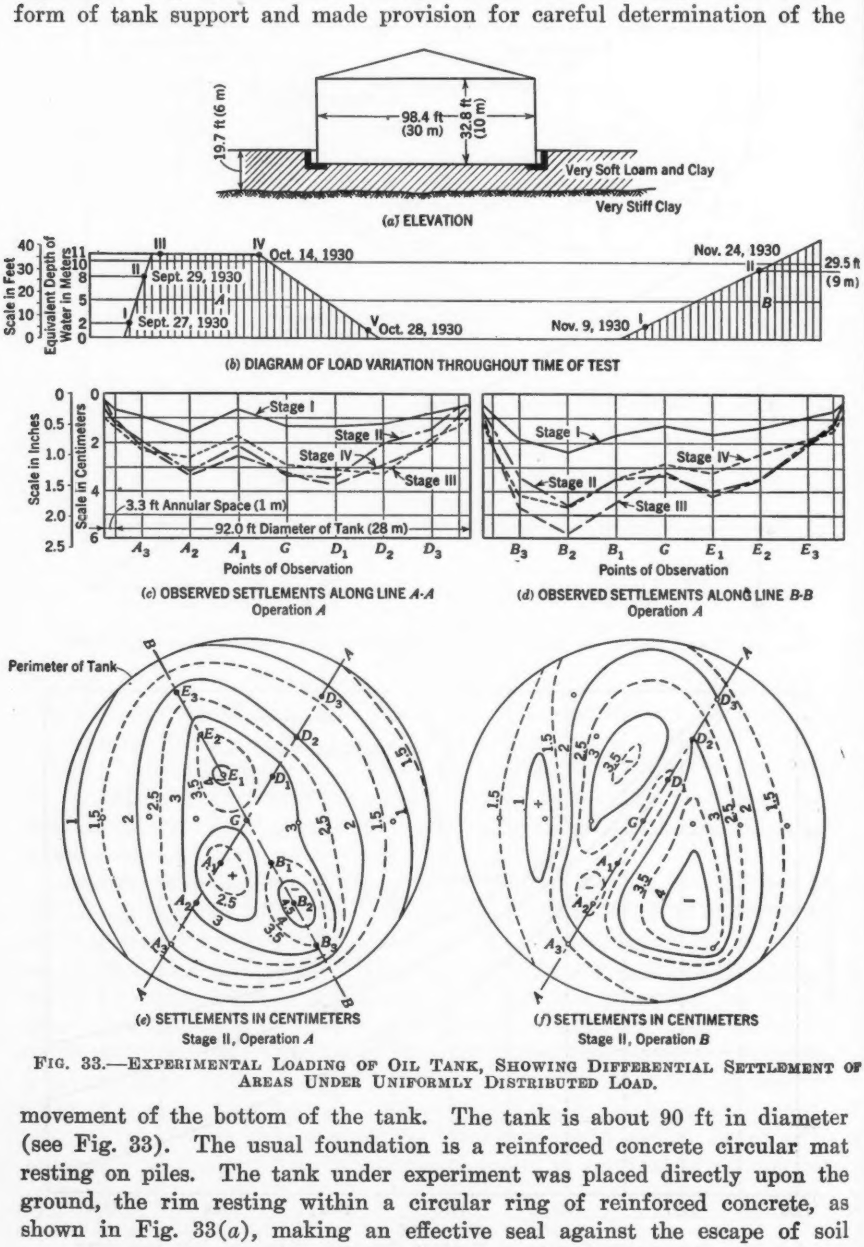


FIG. 33.—EXPERIMENTAL LOADING OF OIL TANK, SHOWING DIFFERENTIAL SETTLEMENT OF AREAS UNDER UNIFORMLY DISTRIBUTED LOAD.

movement of the bottom of the tank. The tank is about 90 ft in diameter (see Fig. 33). The usual foundation is a reinforced concrete circular mat resting on piles. The tank under experiment was placed directly upon the ground, the rim resting within a circular ring of reinforced concrete, as shown in Fig. 33(*a*), making an effective seal against the escape of soil

around the rim. The tank was first filled with water, giving a uniform load of about 1 ton per sq ft. Later, it was emptied and then filled with molasses, giving a load of about $1\frac{2}{3}$ tons per sq ft. Data are not yet available covering the consolidation effect, but the deflections of the bottom are shown in Fig. 33(*c*), Fig. 33(*d*), Fig. 33(*e*), and Fig. 33(*f*). The departure of the actual observed contours from the expected simple dome-like warping of the bottom may be explained, as follows: The major part of the subsidence was due to lateral flow of the soil from the central part of the elevated stratum toward the rim. This lateral flow is essentially responsible for the fact that the peripheral part of the tank scarcely settled. Furthermore, the hills or upward bumps in the bottom (marked thus: +, in Figs. 33(*e*) and 33(*f*))

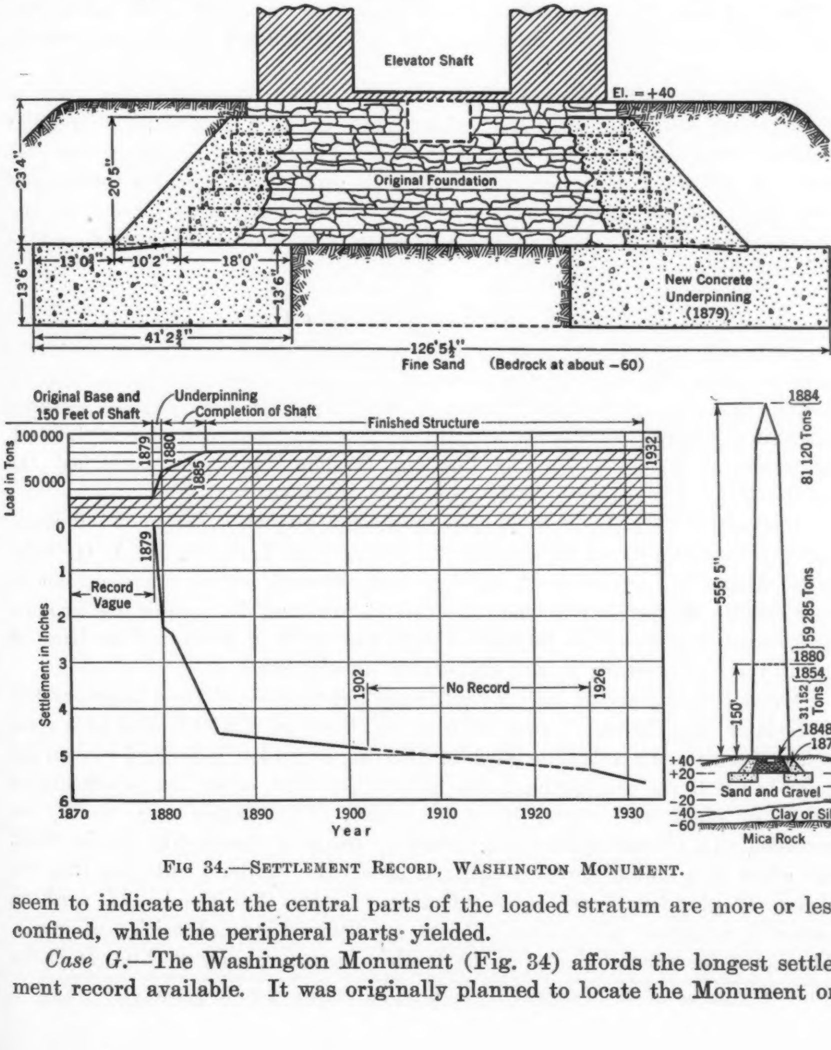


FIG 34.—SETTLEMENT RECORD, WASHINGTON MONUMENT.

seem to indicate that the central parts of the loaded stratum are more or less confined, while the peripheral parts yielded.

Case G.—The Washington Monument (Fig. 34) affords the longest settlement record available. It was originally planned to locate the Monument on

the intersection of the north and south, and east and west axis of the City of Washington, but as the site was swampy, it was actually built on a sandy mound near-by.

The original foundation was only 80 ft square, and when work ceased in 1854, with the structure at a height of 150 ft, the total weight was 31 152 tons, a load of approximately 5 tons per sq ft.

No settlement records prior to 1879 are available, but the original foundations were deemed inadequate by the late Lt.-Col. Thomas L. Casey, Corps of Engineers, U. S. A. (later, General Casey), who resumed operations. The old foundations were underpinned to an area of 126 ft 5½ in. square, in a most skillful manner, and the Monument was raised to its full height by 1884. During the process of increasing the load to 5 tons per sq ft again, the Monument settled 4½ in., subsequent to 1885, another inch, and is probably still slowly settling.

It was originally assumed by General Casey that the underlying stratum was a solid bed of gravel, but borings made in 1931 disclosed an irregular bed of compressible clay overlying the rock, and varying in thickness from 10 to 40 ft. The compression of this layer is probably responsible for most of the observed settlements. The sand and gravel deposits are irregular and of recent origin. Fortunately, the Monument settled quite uniformly, and the structure is nearly plumb.

Photo-Elastic Research

Although the photo-elastic method of investigating the stresses in transparent plates of glass, cut to various shapes and loaded, has been used for many years—great advances have been made recently through the improvement of apparatus and the use of celluloid and bakelite. A leader in the work is Professor E. G. Coker, of London, England, who has published the results of his researches in book form[5] and otherwise.

Under the direction of A. H. Beyer, M. Am. Soc. C. E., of the Civil Engineering Department of Columbia University, New York, N. Y., A. G. Solakian, Assoc. M. Am. Soc. C. E., has made several interesting experiments to visualize the stresses acting in a foundation and its supporting ground. The controlling idea was to make visible the "bulb of pressure," or lines of equal vertical pressure in the supporting underground.

By directing a strong beam of polarized light, obtained from large crystals of Iceland spar, through a sheet of bakelite (0.308 in. thick) loaded by a rigid block of steel or by a flexible block of bakelite, an image is formed containing color bands if the originating beam is of white light, or an image containing bands of black and white, if of one color. This image may readily be recorded on a photographic plate. Such an image is shown, Fig. 35, in which the effect of a simple spread footing is observed. Without entering into the theory of photo-elasticity it is sufficient to state that the bands, working outward, represent diminishing shears or diminishing intensities of stress. They have the general shapes of the "bulb" of pressure. By working from the

[5] "A Treatise on Photo-Elasticity," by L. N. G. Filon and E. G. Coker.

basic theory that the bands which correspond to shears are proportioned to one-half the difference of the two principal stresses, the actual stresses can be computed.

Note that in Fig. 35, the several small "bulbs" blend into one large one below. The large bulb, if it occurs in material capable of consolidation,

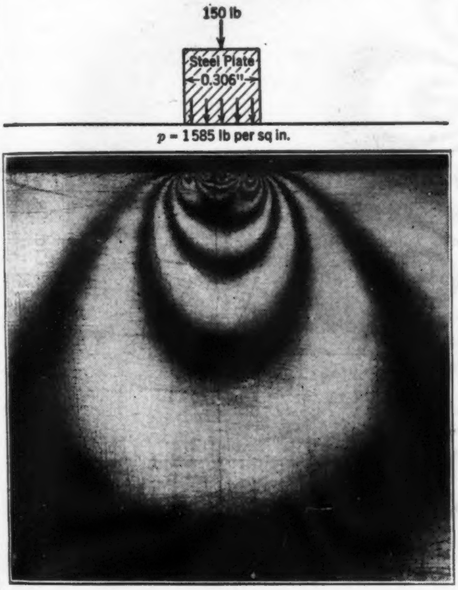


Fig. 35.

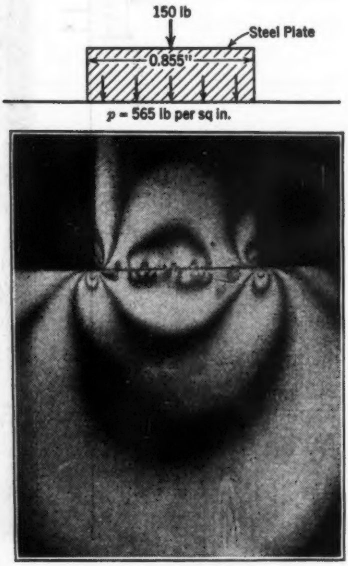


Fig. 36.

would compress and control the settlement of the footing, both as to rate and amount. The bulbs indicated are comparable with those developed in the mechanics of pressure distribution and again illustrate why non-uniform settlement occurs, since the pressure between two surfaces is far from uniform.

Fig. 36 illustrates a case in which bakelite is pressed against bakelite. The upper plate represents a rigid footing—the lower plate the elastic underground or supporting medium. Although both plates were carefully tempered, there are some irregularities on the edges. However, this is as nearly a perfect elastic material as is known. Note that at a short distance from the contact the loads are regular. Note, also, that the non-uniform pressure extends upward into the footings.

To illustrate these phenomena a flexible footing (Fig. 37) was prepared. This is similar to the ordinary individual footing carrying one column or concentration. Note the great variation in pressure—both in the supporting medium and in the footing itself. The slight flexure of the footing contributed largely toward concentrating the load at the center. Fig. 37 plainly indicates that the actual stresses existing both in the footing of this type and in the underground are radically different from those assumed to exist by the ordinary assumption of uniform pressure distribution.

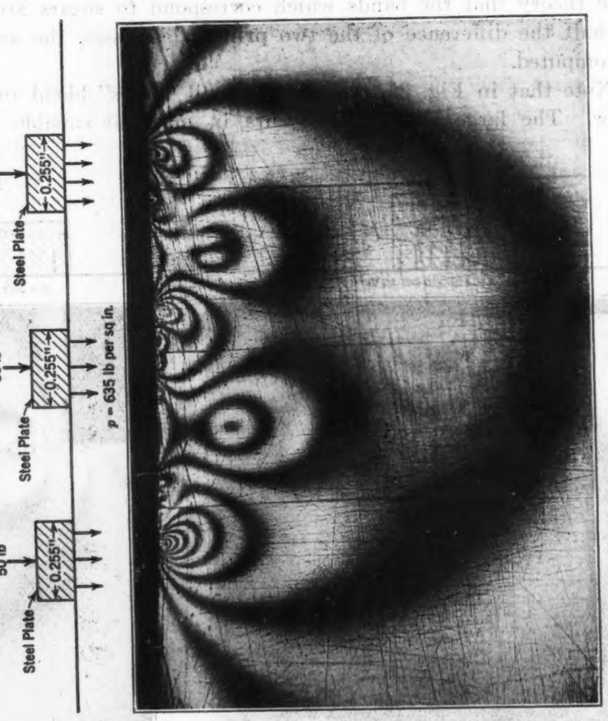

50 lb
0.255"
Steel Plate
50 lb
0.255"
Steel Plate
50 lb
0.255"
Steel Plate
p = 635 lb per sq in.

FIG. 38.

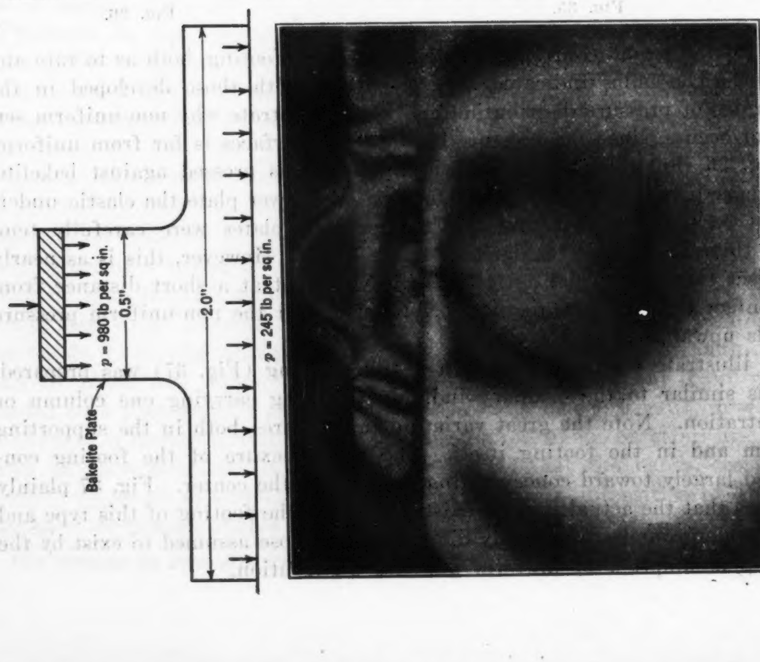

Bakelite Plate
p = 980 lb per sq in.
0.5"
2.0"
p = 245 lb per sq in.

FIG. 37.

Fig. 38 was arranged to show the distribution of stresses from a multiple-footing to the supporting underground. The three steel plates represent three isolated footings. Note that the pressure bulbs immediately below the steel plates are reflected into the space between, so that the reflected bulbs are larger and deeper than the originating bulbs and act in a similar manner. All the upper bulbs, including primary and reflected bulbs, act together and create in the underground a large bulb which may be called the secondary bulb of pressure. Note that the great variations and discontinuities of the primary bulbs are smoothed out and blended in the secondary bulb of pressure. Because of its great depth the secondary bulb affects a large mass of

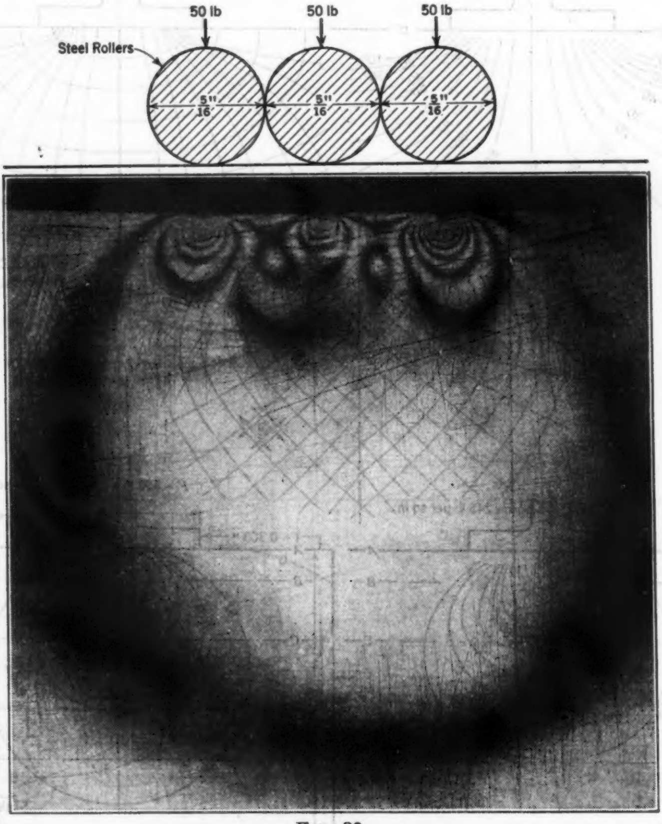


FIG. 39.

material and, therefore, if the underground is compressible the secondary bulb is the controlling factor in the settlement of the structure erected on the compound or mat footing. Heretofore, so much attention has been devoted to investigating the variation in pressure or stress immediately below the footings that the main factor in settlement, the deep underlying stress formation, had been overlooked. Theory and experiment both indicate that there is a

zone of discontinuity for certain depths below a single or multiple footing. The movement and compressions that occur in this region are the cause of quick settlements.

Fig. 39 was arranged to show the distribution of stresses from point applications. It indicates results similar to those of Fig. 37, but rollers are not as good as plates. Figs. 37 and 38 both have application to pile footings. In the case of Fig. 37 it is possible that two footings instead of three would be ample to distribute the load and would lead to the same secondary bulb

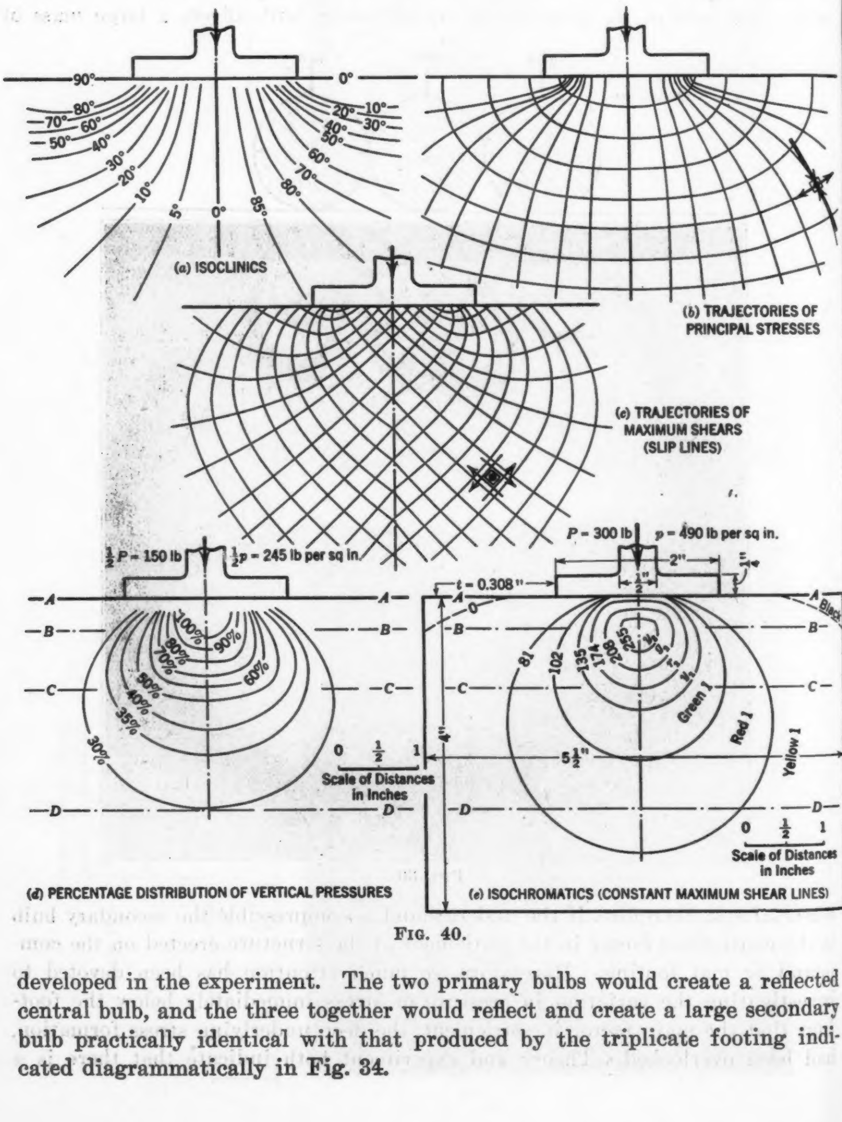


FIG. 40.

developed in the experiment. The two primary bulbs would create a reflected central bulb, and the three together would reflect and create a large secondary bulb practically identical with that produced by the triplicate footing indicated diagrammatically in Fig. 34.

Fig. 39, as applied to a pile footing, indicates that close spacing of piles beneath a structure does not add to the total bearing value of the underground; nor does it increase the size of the large supporting secondary bulb,

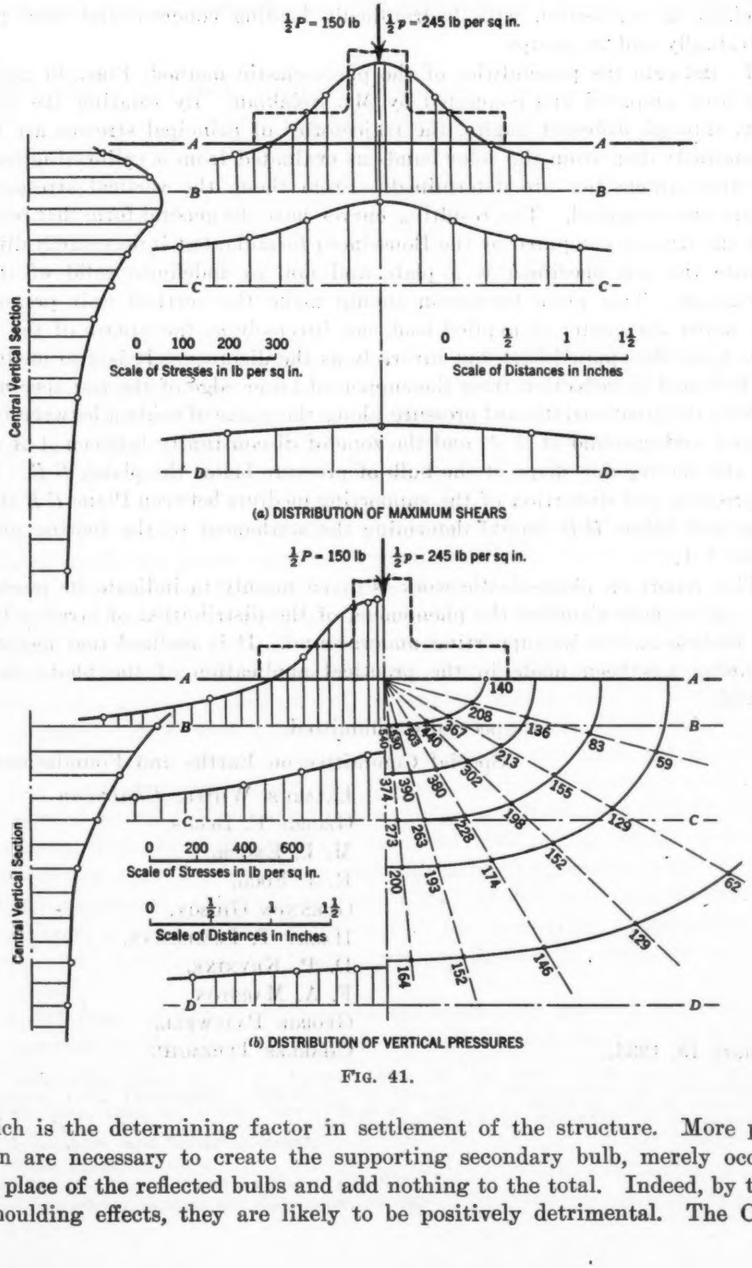


FIG. 41.

which is the determining factor in settlement of the structure. More piles than are necessary to create the supporting secondary bulb, merely occupy the place of the reflected bulbs and add nothing to the total. Indeed, by their remoulding effects, they are likely to be positively detrimental. The Com-

mittee has not had the time nor the resources to go into the question of the proper spacing of pile footings, but all its work indicates that such footings are often improperly used and that the spacing of the pile units is often entirely too close. This is strongly supported by the many tests made commercially in connection with hydraulically loading concrete and steel piles individually and in groups.

To indicate the possibilities of the photo-elastic method, Figs. 40 and 41 have been prepared and computed by Mr. Solakian. By rotating the Nicol prism through different angles, the trajectories of principal stresses are first determined; then from the color bands as evaluated from a calibrating beam, the stress intensities are determined. From them, the vertical stresses or isobars are computed. The resulting curves have the general form that results from the stresses computed by the Boussinesq formula, but it necessarily differs because the test specimen is a plate and not an indefinite solid of three dimensions. This plate formation should make the vertical unit pressures vary under the center of applied load, not inversely as the square of the distance from the applied load, but inversely as the distance. It is also modified and flattened by reflection from the supported lower edge of the test specimen.

Note the great variation of pressure along the plane of contact between footing and underground at *A A*, and the zone of discontinuity between *A A* and *B B* and the regular shape of the bulb of pressure below the plane, *B B*. The compression and distortion of the supporting medium between Plane *B B* and a plane well below *D D* should determine the settlement of the footing above Plane *A A*.

This report on photo-elastic work is given mainly to indicate its possibilities and to help visualize the phenomena of the distribution of stresses both in a footing and in its supporting underground. It is realized that merely a beginning has been made in the practical application of the photo-elastic method.

Respectfully submitted,

Special Committee on Earths and Foundations,

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F. A. MARSTON,
GEORGE PAASWELL,
CHARLES TERZAGHI.

January 18, 1933.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

STRESSES IN REINFORCED CONCRETE DUE TO VOLUME CHANGES

Discussion

By C. P. Vetter, Assoc. M. Am. Soc. C. E.

C. P. Vetter,[23] Assoc. M. Am. Soc. C. E. (by letter).[23a]—The contributions to the discussion of this paper have been most gratifying. Many interesting points have been raised which were insufficiently or less clearly dealt with in the paper. This refers especially to the influence of Poisson's ratio and the effect of a difference in the coefficients of thermal expansion of concrete and steel.

The writer wishes to thank those who have thus, by constructive criticism, helped to make the paper serve its purpose, that is, to throw some additional light on a subject that has been much neglected.

Mr. Slack points out that the thermal coefficients of expansions for concrete and steel are not actually equal, as assumed in the paper. This assumption of equal coefficients of expansion was introduced purposely in order to facilitate the general conception of the subject rather than to simplify the mathematics. Even with this simplification the subject is somewhat intricate and difficult to present clearly. Furthermore, the term, thermal coefficient of expansion (and contraction), when applied to concrete, is not the well-defined quantity Mr. Slack seems to indicate. It changes with the brand of cement, with the richness of the mix, and with the age of the specimen. It is not the same for increasing and decreasing temperatures, nor for wet and dry specimens. Carefully executed tests [24] of dry specimens, 90 days old, have given a mean value of 0.69×10^{-5} for increasing temperature and 0.56×10^{-5} for decreasing temperature, as against a value for steel of 0.67×10^{-5}.

A difference of 0.15×10^{-5} between the coefficients for steel and concrete, as suggested by Mr. Slack, seems high, but even so, the effect on the formulas

Note.—The paper by C. P. Vetter, Assoc. M. Am. Soc. C. E., was published in February, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: May, 1932, by Messrs. Searcy B. Slack, Charles S. Whitney, and M. Hirschthal; and September, 1932, by Messrs. Walter Dreyer, Ralph E. Spaulding, A. H. Beyer and A. G. Solakian, and Lester L. Meyer.

[23] San Francisco, Calif.

[23a] Received by the Secretary March 17, 1933.

[24] *Deutsche Ausschuss für Eisenbeton, No. 23.*

developed in the paper is quite negligible. For example, Equation (12*a*), which gives the ratio of reinforcement required for a temperature drop less than the critical, becomes:

$$p = \frac{S'_c}{S_s - n S'_c - E_s t\,(\epsilon_s - \epsilon_c)} \qquad (43)$$

in which, ϵ_s and ϵ_c denote the thermal coefficients for steel and concrete, respectively. Using the numerical values given by Mr. Slack: $\epsilon_s - \epsilon_c = 0.15 \times 10^{-5}$, $t = 50°F$, and, furthermore, when $S'_c = 300$ lb per sq in., $S_s = 50\,000$ lb per sq in., $E_s = 30.10^6$, and $n = 10$, Equation (12*a*) gives $p = 0.0064$. By Equation (43), $p = 0.0067$. The difference is not impressive bearing in mind that both S'_c and n may vary as much as 25% even in carefully conducted laboratory tests.

Mr. Slack properly draws attention to the considerable effect of a difference between the thermal coefficients in a reinforced concrete unit which is otherwise free to contract. This subject is outside the scope of the paper which deals with restrained units only. Due to the restraint, such units are subject to quite different forces. Mr. Slack's numerical calculations are somewhat approximate. The exact formula for the steel tension in an unrestrained unit is:

$$\phi'_s = \frac{E_s t\,(\epsilon_s - \epsilon_c)}{1 + n\,p} \qquad (44)$$

which, for $p = 0.01$, $t = 50°F$, $\epsilon_s - \epsilon_c = 0.15 \times 10^{-5}$, $n = 10$, and $E_s = 30 \times 10^6$, gives $\phi'_s = 2\,050$ lb per sq in., and for $p = 0.02$, $\phi'_s = 1\,870$ lb per sq in., as against Mr. Slack's 2 250 lb per sq in.

Mr. Whitney lists a number of factors that have not been taken into account, such as plastic flow and variations of the modulus of elasticity and of the coefficient of expansion of the concrete, and concludes:

"The problem is very much more complicated than the author's analysis indicates, so much so that it can not be solved by the formulas presented. No general solution is possible for the variety of structures mentioned."

The writer fully realizes the importance of these factors, which have been disregarded in his analysis. However, it should not be overlooked that these same factors likewise enter into and materially effect every design of every reinforced concrete structure. There is no doubt but that the ordinary methods used in computing "stresses" in reinforced concrete fall far short of the conciseness that engineers are wont to demand when dealing with other materials; yet innumerable structures have been designed by the use of these short-cut methods, not because of ignorance of their shortcomings, but because knowledge of the factors enumerated by Mr. Whitney is as yet so vague that their inclusion in many problems seems a wasted effort, and because the elimination of these factors does not endanger the ordinary structure. When it comes to provisions for shrinkage and temperature variations, not even a pretense of theoretical consideration has been made and (what is far worse), the structures designed by the celebrated rule, "three-tenths of one per cent. for temperature," have failed utterly in this respect, as every

large floor, every dam, every retaining wall, and every flume lining bear witness. The writer has attempted to open this question to discussion from a rational and scientific point of view and it is regrettable that Mr. Whitney did not extend his discussion to show how the factors mentioned by him could be taken into account for more rational design.

Mr. Hirschthal raises a number of interesting points, the discussion of which will help greatly to clarify the problem. He suggests that in a concrete unit restrained in two perpendicular directions the effect of Poisson's ratio will materially reduce the reinforcement required to control cracks in either of the two directions. Unfortunately, this is not so. Let it be assumed, as suggested by St. Venant, that failure of a material in tension takes place when the elongation in one direction has reached a critical value. Let it further be assumed that the unit considered is both infinitely long and infinitely wide, or that it is equally restrained and equally reinforced in two directions. Then the stresses in the two directions must also be equal at a given moment and the elongation of the concrete per unit length, e_c, for a given stress, f'_c will be expressed by:

$$e_c = \frac{1}{E_c} f'_c \frac{m-1}{m} \quad \text{.....(45)}$$

in which, m is Poisson's ratio. Equation (1) then becomes:

$$\frac{f'_s}{E_s} = z - \frac{f'c \dfrac{m-1}{m}}{E_c} \quad \text{.....(46)}$$

and Equation (2):

$$f'_s = zE_s - n\frac{m-1}{m} f'_c \quad \text{.....(47)}$$

Equation (3) and Equation (4) remain unchanged, and Equation (5) becomes:

$$L = \frac{(f'_c)^2}{n\,p^2\,q\,u\,\left(z\,E_c - \dfrac{m-1}{m} f'_c\right)} \quad \text{.....(48)}$$

In going from Equation (48) to the equivalent of Equation (11e), it must be remembered that the stress at which fracture takes place is expressed by $S'_c \dfrac{m}{m-1}$, if S'_c is the breaking strength determined on the testing machine; therefore,

$$L = \frac{(S'_c)^2\left(\dfrac{m}{m-1}\right)^2}{n\,p^2\,q\,u\,(zE_c - S'_c)} \quad \text{.....(49)}$$

and Equation (9) becomes:

$$f_s = f'_c\left(\frac{1}{p} + n\frac{m-1}{m}\right) - z\,E_s$$

from which,

$$p = \frac{f'_c}{f_s + zE_s - n\,\dfrac{m-1}{m}\,f'_c} \quad \text{.....................(50)}$$

which, in turn, for $f_s = S_s$ and $f'_c = \dfrac{m}{m-1}\,S'_c$, becomes:

$$p = \frac{S'_c\,\dfrac{m}{m-1}}{f_s + z\,E_s - n\,S'_c} \quad \text{.....................(51)}$$

which corresponds to Equation (10). Inserting in Equation (51) $f_s = S_s$ and $z = \dfrac{S'_c}{E_c}$, which is the value that in Equation (49) makes $L = \infty$, Equation (51) becomes:

$$p = \frac{S'_c\,\dfrac{m}{m-1}}{S_s} \quad \text{.....................(52)}$$

which is equivalent to Equation (11*a*).

The value of *m* for concrete is not well established, but it is probably about 4; therefore, in a reinforced concrete unit restrained in two directions the reinforcement in each direction for shrinkage should be 33% greater than would be the case in a unit restrained in only one direction.

By similar reasoning, for temperature drop, Equation (12*e*) takes the form:

$$L = \frac{(S'_c)^2\left(\dfrac{m}{m-1}\right)^2}{n\,p^2\,q\,u\,(E_c\,\epsilon t - S'_c)} \quad \text{.....................(53)}$$

and Equation (12*a*), the form:

$$p = \frac{S'_c\,\dfrac{m}{m-1}}{S_s - n\,S'_c} \quad \text{.....................(54)}$$

By similar manipulation of Equations (25) to (30), inclusive, it will be seen that Equation (27) remains unchanged and Equation (30) becomes:

$$T = \frac{S_s - \dfrac{S'_c\,\dfrac{m}{m-1}}{2\,p}}{\epsilon\,E_s} \quad \text{.....................(55)}$$

and, therefore, Equation (12*c*) becomes:

$$p = \frac{S'_c\,\dfrac{m}{m-1}}{2\,(S_s - T\,\epsilon\,E_s)} \quad \text{.....................(56)}$$

It appears to be a general rule that in a structure, subject to shrinkage and temperature drop, which is continuous or restrained in two directions, the reinforcement in each direction should be $\frac{m}{m-1}$ times the reinforcement required in structures that are restrained in only one direction. This ratio has the approximate value 1.33.

Mr. Hirschthal further suggests that shrinkage and temperature stresses in a concrete unit may be influenced by the restraint exerted on the unit by columns, piers, counterforts, and by friction against the foundation. The writer cannot believe that such restraining influences can have any material effect on concrete structures such as those discussed in the paper. Before any cracks have formed, there is no movement of any points of the unit. In the case of shrinkage the shortening of any element equals the elongation due to tension, and the same is the case for temperature drops. When the tension reaches the breaking strength of the concrete, a crack will form and the concrete elements near the crack will move an infinitely small distance away from it. Aside from these small elastic movements, which change directions on either side of the crack, there is no movement of the unit as a whole. It is inconceivable that friction, for instance, can have any appreciable influence on movements of this kind; it would be analogous to say that a steel rod in a testing machine would show a measurably higher strength because it was surrounded by sand which exerted friction against the rod. However, the restraining influences mentioned by Mr. Hirschthal are of considerable importance in another respect in that they cause a concrete unit, which is not infinitely long or not absolutely restrained, to follow approximately the same laws as those established for very long units. In a road pavement provided with expansion joints, for example, the friction may cause a comparatively short unit between joints to act as a restrained unit, and if it is insufficiently reinforced it may crack between the expansion joints.

In his closing remarks, Mr. Hirschthal emphasizes the importance of curing concrete structures in order to minimize shrinkage; the writer fully concurs in this, but the statement that curing for 10 days will reduce shrinkage to about one-fourth, is at variance with the data obtained by other investigators. Fig. 18 gives results of a representative series of tests.[24] Based on these tests it would require curing under water for about 80 days to obtain the reduction in shrinkage that was obtained by Mr. Hirschthal in 10 days. The writer would hazard the guess that Mr. Hirschthal's specimens have not been under observation for a sufficient length of time.

Mr. Dreyer makes a most valuable contribution to the discussion of shrinkage and temperature cracks. The series of tests conducted by the Pacific Gas and Electric Company is the only known rational attack on that much cherished belief that 0.3% reinforcement will prevent cracks in concrete. Mr. Dreyer's concluding remarks: "As a result of this test it was concluded that at least 0.65% of longitudinal steel was required for satisfactory construction," are most gratifying, and the writer believes that had other organizations and individuals had the courage to face the issue in the

same manner, considerable expense and the construction of many "eyesores" would have been avoided.

Mr. Spaulding discusses in some detail the effect of shrinkage on an unrestrained reinforced concrete unit for the behavior of which he establishes

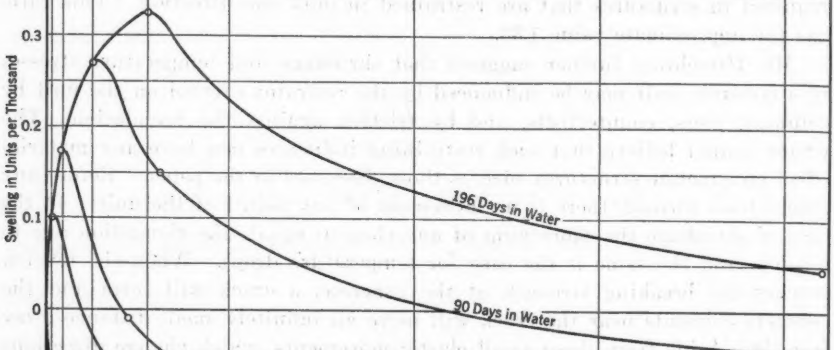

FIG. 18.—SHRINKAGE OF CEMENT MORTARS, 1:3, SUBJECTED TO WATER CURING FOR VARIOUS LENGTHS OF TIME.

Equation (39). Contrary to the opinion expressed by Mr. Spaulding, this equation can be obtained directly from the writer's Equation (9), by making $f_s = 0$, thus expressing that the unit is unrestrained. Equation (39) may also be written:

$$f'_c = \frac{z\,E_c}{\frac{1}{np} + 1} \quad \text{..........................(57)}$$

in which form it was given, about twelve years ago, by Professor Emil Mörsch.[25] Since Professor Mörsch's treatment is original, complete, and generally known, the writer saw no reason for including the treatment of unrestrained units in his paper.[26]

Mr. Spaulding goes on to investigate the effect of shrinkage on a reinforced concrete floor-slab. Although the writer fully agrees with the follow-

[25] "Der Eisenbetonbau," Stuttgart, 1920, p. 127.

[26] For reference to Prof. Mörsch's work on this subject, see *Proceedings*, Am. Soc. C. E., February, 1932, p. 198.

ing general statement of Mr. Spaulding: "It [the shrinkage] increases the negative moments over supports, and, of course, decreases the positive moments, causing much more of the load to be carried by cantilever action," he does not fully agree to the numerical values assigned by Mr. Spaulding to this effect of shrinkage. The shrinkage will produce a constant negative moment throughout the length of the slab to which should be added, algebraically, the moments produced by external loads. The enumeration of this constant negative moment meets with certain difficulties, and the writer believes that it will be necessary to proceed along a path somewhat different from that suggested by Mr. Spaulding.

Messrs. Beyer and Solakian have furnished, most successfully, experimental proof of one of the fundamental assumptions upon which the paper was based. By photo-elastic methods they examined the stresses in a bakelite unit reinforced with an aluminum rod. Their tests show conclusively that the bond stresses are confined to the ends of the specimen and that, near the middle, the stresses in the bakelite and aluminum rod are constant. That the increments per unit length of longitudinal stresses near the ends are not constant, such as assumed in the paper, need not be cause for worry. The bond characteristics between bakelite and aluminum need not even approximate those between concrete and steel. In this connection it is interesting to compare the results obtained by Messrs. Beyer and Solakian with those of Mr. W. H. Glanville for the Department of Industrial and Scientific Research, in London, England. With the aid of an elaborate extensometer arrangement Mr. Glanville measured directly the distribution of the strain in a steel rod encased in a concrete unit subject to shrinkage. One of his diagrams is shown in Fig. 19. It is seen that the increment per

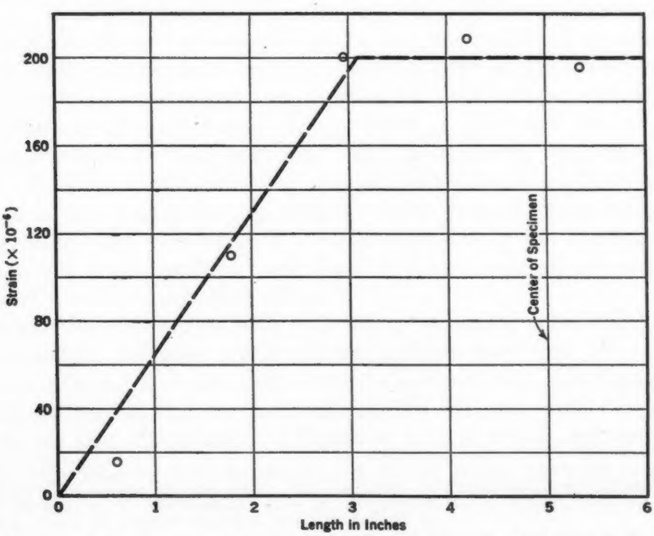

FIG. 19.—STRAIN IN A STEEL ROD ENCASED IN A CONCRETE UNIT SUBJECT TO SHRINKAGE (FROM TESTS BY W. H. GLANVILLE).

unit length of strain in the steel rod is almost exactly constant near the end of the specimen and zero near the middle.

More puzzling in the tests by Messrs. Beyer and Solakian is the distribution of the longitudinal stresses, σ_x, over the cross-section of the bakelite (see Fig. 16). It seems difficult to understand why, near the ends of the unit, the tension should be the greatest at a distance farthest from the reinforcement rod. In this experiment the aluminum rod is the stress-producing agency and it would be expected that the ends of the specimen would be slightly conical with the apex at the center of the rod. Since the strain is greater near the rod, the stress should likewise be greater at this point.

It would have made an interesting check on the stress distribution had the actual shortening of the aluminum rod been measured directly.

The writer learns with the greatest regret that Mr. Meyer, who had also contributed to this discussion, passed away after a short illness on December 27, 1932. Mr. Meyer's discussion was clear-cut and to the point. He contributed the results of tests conducted by the Los Angeles Water Department, from which he deducted some interesting conclusions. It is evident that in Mr. Meyer the profession has lost an able investigator.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

THE COMPENSATED ARCH DAM

Discussion

BY A. V. KARPOV, M. AM. SOC. C. E.

A. V. KARPOV,[8] M. AM. SOC. C. E. (by letter).[8a]—These seems to be a rather large school of highly theoretical engineering thought that is based on the postulate that assumptions should be made to coincide with theoretical ideas, whether or not they are in accord with practical conditions.

The discussion by Professor Kalman is based on such a postulate. The last sentence of his discussion is all that is necessary to bring out the fundamental differences between him and the writer. The proper answer to this objection is that it has nothing to do with the subject under discussion. No matter if it suits or does not suit one's theoretical ideas, the cold fact is that no commercial arch dam was ever built as a monolithic structure; and it can never be built as a monolithic structure as long as engineers are limited to present materials and methods of construction. If there is any doubt on this point, it is only necessary to observe the behavior of the vertical construction joints at the up-stream and down-stream faces of an arch dam at different loadings. Such a dam can be built only as a number of adjoining loose blocks or piers, and the question at issue is how to make the best design under such limitations. The purpose of the paper was to give the theoretical answer to this question. The answer may be summarized as follows: Keeping the thickness of the horizontal arches uniform, and shaping them so as to suit the topography of the canyon, it is possible, theoretically, to build a dam without bending moments in the horizontal arches, if an infinitely large number of very small V-shaped vertical joints is properly distributed over the horizontal arches.

Professor Kalman agrees with that theoretical answer, but thinks that the methods used in the paper are too formal and cumbersome. He proves by a number of different methods, that t must be constant and that Equations (67) and (68) are correct, but he invariably prefers his own methods. Correctly, he points out that the solution of Equation (71) is wrong and, consequently,

NOTE.—The paper by A. V. Karpov, M. Am. Soc. C. E., was published in April, 1932, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: October, 1932, by Messrs. R. A. Sutherland, Lars R. Jorgensen, L. T. Evans, and H. W. Sibert; and March, 1933, by Eugene Kalman, M. Am. Soc. C. E.

[8] Designing Engr., Hydr. Dept., Aluminum Co. of America, Pittsburgh, Pa.

[8a] Received by the Secretary April 19, 1933.

Equations (71) to (85), inclusive, must be corrected. His statement that $\int_0^s \beta \, ds = 0$, holds true only as long as the foundation deflections are such as not to cause rotation of the abutments. Although he already has proved Equation (68) by his own method, he questions Equation (70), stating (see following Equation (117)): "The equality of the definite integrals of two functions between two given limits does not imply the identity of the functions." That is a correct mathematical statement, the only objection being that if it is to apply to the derivation of Equation (70) it should be brought to a logical conclusion, as follows: "But if these two definite integrals are identical at each value of the limits, then the functions are identical."

The reasoning by which Professor Kalman compares the idealized and actual methods in which an arch deforms is a proper general statement, but has very little to do with a compensated arch dam. No matter what the intermediate conditions are, the stresses and deflections of an arch dam are defined exactly by the final conditions, provided the elastic limit is not overstepped and the deformation proceeds without slippage.

In an ordinary dam, considerable shearing forces are developed in the horizontal arches, that may result in two adjoining blocks slipping one with respect to the other. In a compensated dam, properly shaped, the development of shearing forces of any magnitude is practically excluded. Of course, the uniformity with which the hydraulic load is applied, is an additional factor that makes improbable any slippage in a compensated dam.

In his discussion of the practical aspects of compensation, Professor Kalman approaches the subject in a highly theoretical manner. Apparently, he thinks that there are definite limits to the size of the vertical **V**-shaped joints; but from a practical point of view, there are no difficulties with respect to their construction. An open joint was assumed in the original paper, in order to simplify the theoretical proof, but it was also stated that in the actual construction of the dam closed joints are proposed and elastic fillers are to be used. Fundamentally, it simply means that the dam is to be composed of two kinds of materials—one, the concrete of the main body, having a high modulus of elasticity, and the other, the **V**-shaped inserts, having a lower modulus of elasticity. By varying the thickness of the inserts and the modulus of elasticity of the insert material, any theoretically desirable thickness of joint may be constructed.

One method to accomplish this result would be to use sheets of compressible material of varying thickness in the vertical joints between adjacent blocks. For instance, if tar paper is used, the thickness of the joint filler can be changed by varying the number of sheets, and after the dam is loaded the residual filler will act as a part of the dam. Regarding compensation from this point of view, it would seem unnecessary to go into the involved speculations which are used by Professor Kalman to prove the impracticability of compensation owing to the impossibility of forming thin **V**-shaped joints.

The entire matter of compensation should be treated like many other engineering problems, on the basis of a reasonable compromise between

theoretical and practical requirements. Theoretically, to have perfect compensation, an infinitely large number of **V**-shaped joints would be required. Practically, however, a limited number will reduce the bending moments so as to make them negligible, the moments being zero at the joints, with the possibility of having limited moments in the part of the horizontal arches between the joints.

Professor Kalman regrets that the original paper does not contain more about the actual design of a dam for a given canyon section. The writer believes that it is undesirable to include in a theoretical paper (which has a definite purpose and which is necessarily limited in length), an extended practical application of the problem. Such application should be the subject of a separate paper.

Mr. Evans and Professor Sibert do not agree that a uniform thickness of horizontal arches is a theoretically necessary condition. Mr. Evans is in error when he derives Equation (93); the moment, M_{xy}, at the point, x, y, is due to all the loads applied at the up-stream face of the arch and, as clearly shown by Equation (8), must be a summation of the moments due to each of the forces, p'.

Professor Sibert is in error when he derives Equation (101) by integration of Equation (100). This equation cannot be integrated in its general shape, because p' is a function of x' and y'.

The foregoing covers also the points raised by Mr. Sutherland.

Mr. Jorgensen raises an interesting question, relating to the "water-soaking effect" and the differences in behavior between wet and dry concrete. The writer believes that in an arch dam a number of important secondary influences must be considered, but nevertheless the prime factor is the elastic behavior of the dam and its foundations. A rational design is possible only if full consideration is given first to the prime factor—that is, the elastic behavior—and next, if corrections are made to cover the influence of secondary factors.

Mr. Jorgensen's reasoning leads to the conclusion that, since enough is not known about the water-soaking effect, the fundamental elastic behavior of the vertical elements of the dam, as well as the foundations, should be neglected. Such a conclusion can scarcely be defended from a theoretical point of view. From a practical standpoint, the introduction of a phantom design by assuming that the dam acts as a number of independent horizontal arches, is a short-cut that results in a decidedly inferior structure.

The experimental study of the water-soaking effect is of considerable interest, and such study could be made by comparing the behavior of an actual arch dam and a model made of material that excludes the possibility of the water-soaking effect. It is expected that such a comparison would give at least some quantitative idea about the influence of this effect.

In conclusion, the theoretical status of the compensated arch dam is not changed. The discussion centers on the objections in the practical application. It is only natural that an attempt to introduce radical changes in accepted design methods will meet with considerable opposition, but if the theoretical foundation is proved, the first step is made in the practical application of the method proposed in the paper.

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DISCUSSIONS

GEORGE WASHINGTON BRIDGE GENERAL CONCEPTION AND DEVELOPMENT OF DESIGN

Discussion

By O. H. Ammann, M. Am. Soc. C. E.

O. H. Ammann,[17] M. Am. Soc. C. E. (by letter).[17a]—The writer appreciates very much the valuable contributions and the complimentary comments on his paper. The excellent exposé of the fundamentals in bridge design by Mr. Howard, and his recognition of their application to the George Washington Bridge are gratifying. Undoubtedly, like the writer, Mr. Howard has struggled to extricate himself from the maze of structural conceptions which again and again appear to grip the imagination of the engineer and lead him to devise complicated, and sometimes abortive, solutions.

Time is an important factor in the mature development of the design of an important structure, but, unfortunately sufficient time is often lacking. Structures have been put on paper hurriedly, sometimes not even to scale, and from that point constructed with amazing speed in the field, with no thought of æsthetic conception or refinement in design. It is to be hoped that the era of driving prosperity will have taught a lesson in this respect and that in the future engineers will be able to proceed more deliberately in the conception and moulding of important structures.

Mr. Howard wonders where 50 000 000 more vehicles to cross the Hudson River annually will operate in Manhattan. This figure appears impressive, but when this is compared with the number of annual vehicle trips to and from Manhattan across the Harlem and East Rivers, which is more than 200 000 000, and with the still vastly greater number of vehicle trips localized in Manhattan, the figure becomes almost insignificant.

Moreover, unlike the commuter traffic, which pours into Manhattan during a few hours in the morning and leaves it similarly in the evening, the

Note.—The paper by O. H. Ammann, M. Am. Soc. C. E., was published in August, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: November, 1932, by E. E. Howard, M. Am. Soc. C. E.; January, 1933, by C. T. Schwarze, M. Am. Soc. C. E.; and February, 1933, by Messrs. Gustav Lindenthal and Harold M. Lewis.

[17] Chf. Engr., The Port of New York Authority, New York, N. Y.

[17a] Received by the Secretary April 11, 1933.

vehicular traffic across the Hudson River is more uniformly distributed, and almost throughout the day one vehicle leaves Manhattan to every one that comes in. Additional Hudson River crossings tend to increase traffic circulation; they not only impose new traffic on the streets of Manhattan, but they also relieve traffic "bottled up" there.

The increase in number of vehicles across the Hudson River will come not so much from increase in population, as Mr. Howard assumes, but by far the greater proportion will come from a spread of population to the suburbs and from encouragement of more frequent use of motor vehicles, both induced by better highway facilities. These facilities for traffic in and out of Manhattan in all directions are yet far from adequate and economical to meet the demands when normalcy returns. This applies particularly to through, or long-distance, traffic which, while yet relatively unimportant, as Mr. Howard properly recognizes, is bound to assume voluminous proportions in course of time.

Mr. Howard's recognition of the importance of adequate highway approaches cannot be appreciated too strongly. Developments in the future will justify even more elaborate and advanced arrangements than those that had to be adopted for the George Washington Bridge under prevailing conditions.

The writer is encouraged by the emphasis laid upon the æsthetic side of bridge design by Professor Schwarze and his acknowledgement of the efforts in this respect in the case of the George Washington Bridge. Conditions, more particularly the naturally imposed length and proportions of spans and their relation to topography, were not conducive to rendering this bridge a structure of outstanding beauty. Whatever æsthetic merits the George Washington Bridge possesses are, in the writer's opinion, due largely to the structural simplicity and functional clarity of the structure as a whole. The design of the towers, which has called forth varied views and comments is of secondary æsthetic effect. Undue weight has been attached to it in some criticisms. As may now be fully appreciated by viewing the bridge from a distance, in its setting in the landscape, the proposed covering of the steel skeleton with masonry would scarcely alter the picture and the writer, judging from the architect's perspectives, does not share the concern of Professor Schwarze that such encased towers might appear too slender and weak. The outside proportions of the towers, whether they were to remain bare or were to be encased, were purposely selected with a view to expressing their function of carrying an enormous load, but without producing undue massiveness. Preference for the steel towers as they stand, or for encased towers as originally proposed, is entirely one of individual taste.

The writer is indebted to Mr. Lindenthal for his very interesting and valuable contributions to the history of bridging the Hudson River with which he has been so intimately connected and which centers itself largely around his own untiring efforts.

Mr. Lewis calls particular attention to the suitability of the George Washington Bridge for railroad services in accordance with the early studies

embodied in the Regional Plan of the City of New York and Its Environs. The writer is in entire sympathy with this idea and provisions in the design of the bridge will make it feasible to carry it out, at least to the extent of providing four, possibly six, rapid transit tracks. The extent to which bus passenger traffic over this bridge is constantly increasing in volume is evidence of a demand for this service, which eventually, however, will assume proportions that cannot be served effectively except by rapid transit.

The diagram of vehicular traffic across the Hudson River presented by Mr. Lewis (Fig. 26) illustrates a phenomenon that has been manifested prior to the present economic depression in almost every instance of a new transportation artery in and around New York City, namely, that estimates of traffic volume remained far behind actual developments. The Port Authority Staff has had to increase its forecast estimates of Hudson River traffic repeatedly. The revision made in 1930, based upon traffic trends at that time, is illustrated in Fig. 8 of the writer's paper. The depression has put a stop to the accelerating increase and its effect will undoubtedly be felt for many years to come. A review of its estimates in September, 1932, has led the Port Authority Staff to forecast traffic that is between the rate shown in Fig. 8 and that in Fig. 26 of Mr. Lewis' discussion. Fig. 27 was prepared in Septem-

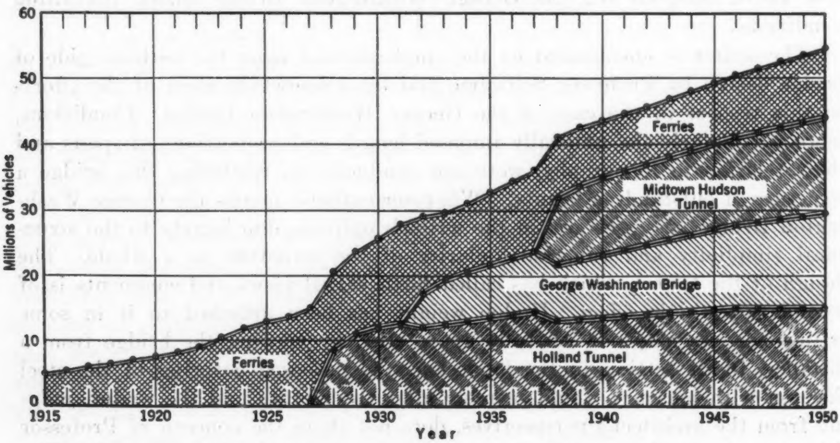

FIG. 27.—RECORDED AND ESTIMATED ANNUAL VEHICULAR TRAFFIC FOR ALL HUDSON RIVER CROSSINGS.

ber, 1932, and should be compared with Fig. 8 of the writer's paper which embodies the traffic estimates made by the Port Authority Staff in 1930. The traffic data represented in Fig. 27 formed the basis of the application of the Port Authority to the Reconstruction Finance Corporation for financing the Midtown Hudson Tunnel at 38th Street, Manhattan, as a self-liquidating project under the Emergency Relief Construction Act of 1932.

The writer agrees with Mr. Lewis that the Regional Plan forecast for 1950 is still conservative and may be regarded as a minimum estimate. The traffic over the George Washington Bridge, as actually developed in 1932, practically agrees with Mr. Lewis' forecast of 6 100 000 vehicles.

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DISCUSSIONS

FORESTS AND STREAM FLOW

Discussion

BY CHARLES H. LEE, M. AM. SOC. C. E.

CHARLES H. LEE,[64] M. AM. SOC. C. E. (by letter).[64a]—The authors have presented a thorough and convincing analysis of the effects of deforestation by cutting, and denudation by fire, respectively, upon the stream flow of two small mountain water-sheds. They have done well in distinguishing between deforestation by cutting and denudation by fire, because the destruction of the forest litter in the latter case constitutes a fundamental difference when considering the effect upon stream flow.

Although much impressed by the conclusions respecting these two water-sheds, the writer does not believe that the same conclusions can be extended indiscriminately to other water-sheds. Justification for this statement appears when it is found that differing causes produced similar results in the two water-sheds under consideration. For example, the writer recently examined precipitation data carefully observed during a 4-year period at eight stations in an area of mixed coniferous and deciduous forest trees, which indicate an average interception of rainfall during the summer months of 22 per cent. This percentage would fully account for the 12% increase in summer stream flow from the Wagonwheel Gap area after deforestation, even allowing for errors due to leaf drainage, etc., since this flow is derived principally from copious summer storms (Table 2). At Fish Creek, on the other hand, summer rainfall is practically zero, and stream flow is derived entirely from ground-water storage. The cause for the 475% average increase in summer stream flow, amounting to 65 acre-ft per annum (Table 9), can here be entirely

NOTE.—The paper by W. G. Hoyt, M. Am. Soc. C. E., and H. C. Troxell, Assoc. M. Am. Soc. C. E., was presented at the Annual Convention, Yellowstone National Park, Wyoming, July 6, 1932, and was published in August, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1932, by C. G. Bates, Esq.; November, 1932, by Messrs. J. E. Willoughby, and A. L. Sonderegger; December, 1932, by Messrs. J. C. Stevens, Harry F. Blaney, Daniel W. Mead, Ralph R. Randell, H. K. Barrows, Donald M. Baker, Ralph A. Smead, and George H. Cecil; February, 1933, by Messrs. C. W. Sopp, and W. B. Rowe; March, 1933, by Messrs. W. C. Lowdermilk, Rhodes E. Rule, and Robert E. Kennedy; and April, 1933, by Messrs. Herman Stabler, and H. S. Gilman.

[64] Cons. Hydr. Engr., San Francisco, Calif.

[64a] Received by the Secretary April 7, 1933.

accounted for by the burning of riparian vegetation which extended along the stream channel for fully ten miles (see Fig. 13). In other words, similar changes in stream flow resulted from entirely different causes. Further search might disclose other instances of apparent but not real agreement in the results for the two water-sheds.

The writer is firmly of the opinion that correct general conclusions cannot be reached by taking into consideration forest and stream-flow data alone, but that all the hydrologic elements which influence stream flow must also be considered. Most of these elements are generally recognized, and may be briefly summarized as follows:

Precipitation upon a water-shed finds disposal from the area either as water-shed losses, stream flow, or sub-surface leakage. Water-shed losses consist of (*a*) evaporation from leaves and branches by interception (immediate); (*b*) evaporation from the soil (within 10 to 18 days after a storm); (*c*) evaporation from snow on the ground (within 2 to 6 months after a storm); (*d*) transpiration from vegetation (during growing season); and (*e*) evaporation from permanent water bodies, if any (continuous). Moisture that supplies transpiration from vegetation is derived from water temporarily absorbed by the soil, which remains within the root zone during the growing season and is in excess of the wilting coefficient of the soil.

Stream flow is derived from immediate surface run-off (largely flood flow), and from water permanently absorbed by the soil which penetrates below the zone of plant roots and joins ground-water storage at the water-table, reaching stream channels later as ground-water discharge (normal or low-water flow). Stream flow as measured at any point in the channel is a net quantity after deduction of channel losses, of which transpiration from riparian vegetation is usually the most important. Channel seepage losses may also be important where the channel is permeable and does not cut the water-table. Sub-surface leakage is a function of geological formation, and with an impermeable bed-rock is negligible.

This complete view of the problem shows that the various physical characteristics of water-sheds, such as meteorology, vegetation, topography, soils, and geology, all have an influence upon stream flow, and also that several physical processes are involved, among which are evaporation, transpiration, and absorption. Of these processes, evaporation and transpiration have and are receiving much attention, while little study has been given to absorption, either as to rate or total quantity.

Absorption may be defined as the processes by which water enters the earth. These processes include flow into sink holes, influent seepage, absorption by capillary films, hygroscopic absorption, and inflow of atmospheric water vapor. Absorption from precipitation has two aspects, its rate, which is directly related to immediate surface run-off and flood flow, and its total quantity, which is directly related to ground-water discharge which maintains normal and low-water flow.

Rate of Absorption.—What bearing does the rate of absorption have upon the relation of forests to immediate surface run-off? Just this, that the rate

of absorption depends primarily upon the permeability of the soil, and in so far as forests increase soil permeability, they decrease immediate surface run-off, and to the extent that they decrease soil permeability they increase such run-off. Forests growing in highly permeable soils free from clay, such as dune sand, pumaceous volcanic deposits, coarse alluvium, sandy loams with less than 4% clay, and even pure silt, such as certain loess deposits, have little or no effect upon surface run-off. Forests growing upon impermeable soils and those which contain more than 4% of clay, however, have an important effect upon surface run-off by decreasing it as a result of increasing the permeability of the soil. They accomplish this through the agency of roots and forest litter by the following processes:

1.—Roots penetrate the ground where they open cracks in hard formations, such as rock, and permeate softer formations, such as clay beds, with root systems that expand the soil and ultimately give channels for the entrance of water.

2.—Forest litter promotes the growth of soil fauna, such as worms, bacteria, etc., which are very effective in opening up an impermeable soil.

3.—Forest litter provides a source for humus, which is of greatest importance in opening a clayey soil, not only mechanically by mingling with inorganic soil particles and separating them, but chemically by the production (in the process of decay) of solutions which, acting in conjunction with natural lime in the soil, facilitate flocculation or the assembling of clay particles into soil crumbs.

4.—Forest litter protects inorganic soil particles from disturbance by the impact of falling rain drops or by the movement of flowing sheet water, and thus prevents the clay from segregating out into suspension with a resultant sealing action upon a soil which, although originally more or less impervious, has been opened up by Processes 1 to 3. As an illustration, a suspension of colloidal clay will completely seal a column of clean medium beach sand having a weight one hundred times that of the clay.

5.—Forest litter protects the surface of the clayey soil from consolidation by preventing the puddling produced by hoofs of grazing animals, or the crusting and consolidation caused by shrinkage with exposure to the sun. The degree of consolidation of a clay soil has a very marked effect upon its permeability.

6.—Finally, forest litter prevents erosion with its destructive effects, not the least of which is the sealing of permeable channels and overflow areas in alluvial filled valleys which are normally the intake areas for the economically important ground-water resources of these valleys. It is to be noted, however, that brush or grass may be as effective as forest litter in preventing erosion.

Total Quantity Absorbed.—The quantity of absorption is dependent not only on soil permeability, but also on the amount and distribution of rainfall during the annual cycle, the storage capacity of the soil in the zone of aeration available for plant discharge, and the transpirational requirements of

vegetation. Of the total quantity absorbed, a portion, which may be termed temporary absorption, reaches only the zone of aeration from which it is drawn back and escapes into the atmosphere by evaporation from moist soil or transpiration from vegetation. The remainder is permanent absorption which penetrates to the water-table and enters the zone of saturation. The latter is the ground-water storage which feeds stream flow.

Soil moisture research with partly saturated soils has shown conclusively that movement of water through capillary films not within the capillary fringe of a water-table is so slow that re-adjustments do not take place during the annual cycle. As a result, downward movement of water through the zone of aeration does not occur unless and until moisture content is at or above that at which free gravity drainage commences.

Tests made in brush-covered areas of sandy and gravelly soils on valley floors in Southern California to determine total deficiency in moisture content within the root zone (8 to 18 ft) at the end of the dry season, indicate values of from 8 to 14 in. The same tests show that, including current losses during the rainy season by soil evaporation and transpiration, approximately 19 in. of seasonal rainfall is necessary before any considerable quantity of water will penetrate beyond the root zone. Data obtained by the U. S. Geological Survey in Central California have been analyzed by the writer, which indicate that for uncultivated fine sandy silt loam soils with natural grass and weeds, a depth of approximately 12 in. is required. No data are known to be available for mountain water-sheds with either brush or forest. The steeper slopes tend to cause surface run-off prior to full satisfaction of field capacity, while the shallower soils tend to reduce the available storage capacity in the soil. It is believed that for most mountain water-sheds the limit is between 12 and 20 in.

It is thus apparent that in a semi-arid region a considerable quantity of rain must fall before permanent absorption can commence. In fact, in regions where the average rainfall is between 12 and 20 in., it is probable that accretions to ground-water storage through deep soils supporting natural vegetation occur only in wet years. It should be recognized, however, that in irrigated areas permanent absorption from rainfall occurs when considerably less than these quantities have fallen; also, that in shallow soils which are less in depth than normal root penetration and are underlaid by bed-rock, or other impervious formation, gravity water may begin to accumulate at the bottom of the soil column and a water-table may form when lesser aggregate quantities of rain had fallen. Under the latter conditions, which are not unusual for mountain water-sheds, ground-water contributions might be made to adjacent stream channels from the superficial soil cover, as distinguished from that yielded by cracks, joints, and fissures in the bed-rock. The quantity thus contributed is controlled by the length, slope, and permeability of the intervening soil path. In view of the very slow movement of sub-surface water, the amount of such contribution is limited to that yielded by the lower slopes, that from upper slopes being intercepted and transpired during the growing season by intervening vegetation.

Turning again to forests, the following applications can be made of these facts regarding the amount of absorption:

(*A*) Under arid conditions, vegetation has no effect upon ground-water contribution to the summer flow of streams, except that roots of riparian, or stream-border, vegetation discharge ground-water that enters or leaves surface channels as influent or effluent seepage, and thus decrease the flow.

(*B*) Under humid conditions, forests and other vegetation growing upon permeable soils decrease ground-water contribution to normal and low-water flow by transpiration, the quantity of the latter depending upon the requirements of the individual species. Deciduous forests consume more than grasses, but evergreens possibly not as much. Forests and brush growing upon soils containing appreciably more than 4% clay, however, offset this decrease by increasing the permeability of the soil, and in clayey soils may actually increase normal and low-water flow.

(*C*) Under semi-arid conditions (wet winter and dry summer), forests and other vegetation growing on deep soil decrease ground-water contributions to summer flow by transpiration as compared with bare soil. As precipitation and melting snow in such regions occur principally during the dormant period when transpiration is very slight, the decrease is less than the transpiration requirements of the species. The measure of the decrease is approximated by the quantity of soil moisture within the root zone and between field capacity and wilting coefficient. This follows from the fact that transpiration supplied by rain which falls during the growing season is largely an offset against evaporation from bare soil during the same period. The moisture depletion varies from 8 to 14 in. for brush on deep sandy soil in valleys. These limits would be less for shallow soils and for deep clayey soils, and are probably not out of line for forests in mountain water-sheds. For a season when rainfall plus current evaporation from bare soil exceeds these amounts during the period of replenishment of soil moisture, the substitution of forests for bare land would have no effect upon summer flow derived from ground-water.

Summarizing this discussion, and with particular reference to absorption:

1.—Forests decrease immediate surface run-off from precipitation: (*a*) By interception and evaporation from foliage, especially in summer; (*b*) by retarding the rate of melting snow lying on impermeable soil or rock; and (*c*) by increasing the permeability and, hence, the rate of absorption of soil containing more than approximately 4% of clay.

2.—Forests have no effect upon immediate surface run-off from precipitation or melting snow when growing upon highly permeable soil, nor under conditions of such long-continued or intense rainfall that the forest litter becomes saturated.

3.—Forests decrease summer stream flow derived from ground-water storage: (*a*) In humid regions by the full amount of seasonal transpiration on highly pervious soil areas and by lesser amounts on soils containing more than 4% clay to the extent that permeability is increased; and (*b*) in semi-arid

regions by a portion of the transpiration not exceeding from 12 to 20 in., with smaller limits under various conditions of soil depth and permeability. This assumes that evaporation by interception offsets evaporation from bare soil without forest.

4.—Forests on clay soil may increase summer stream flow derived from ground-water storage by increasing soil permeability.

5.—Forests decrease total annual run-off as compared with that from bare soil, but as compared with brush or grass they decrease the run-off only when the total of forest transpiration and interception exceeds that of the brush or grass.

While these conclusions do not differ from those of the authors for the two small water-sheds studied, they indicate the errors that might result from too general an application.

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DISCUSSIONS

TESTS OF RIVETED AND WELDED STEEL COLUMNS

Discussion

By M. O. Fuller, M. Am. Soc. C. E.

M. O. Fuller,[17] M. Am. Soc. C. E. (by letter).[17a]—The discussion of this paper has been a tribute to the memory of the late Willis A. Slater, M. Am. Soc. C. E., who was interested in the Society for many years, had contributed papers to its publications, and had taken an active part in discussion.

The tests described by Professor Roark are interesting in that they throw light on the total strains and stresses produced by a given welding process and their effect on the mechanical properties of the member as a whole. The fact that the apparent stresses from the tensile and compressive tests are about the same order of magnitude is very interesting as it may be of service to the designer in indicating the effect of the two kinds of stress upon welded connections.

The points mentioned by Mr. Kirkley are, as he states, of minor importance with respect to the strength of the welded and riveted steel columns, and are usually taken care of by careful inspecton.

Considerable is still to be learned regarding the effect of welding, and investigators are carrying on tests in this field. The work reported in the paper was only a small contribution to this subject, and the writer agrees heartily with Professor Caughey that much needs to be done to establish the relation between the initial tensile and compressive stresses set up on account of unequal shrinkage in cooling.

The question of crookedness in the test columns was one that had not been anticipated and for that reason the original column that was crooked was replaced by another made at a later date. The original column (No. 4) was tested simply as a matter of routine, as it was realized that one test was not sufficient to draw conclusions to any extent. Messrs. Holt and Sturm have

Note.—The paper by the late Willis A. Slater and M. O. Fuller, Members, Am. Soc. C. E., was published in September, 1932, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: January, 1933, by Messrs. Raymond J. Roark, and Lyndon F. Kirkley; and February, 1933, by Messrs. R. A. Caughey, E. G. Walker, and M. Holt and R. G. Sturm.

[17] Associate Prof., Civ. Eng., Lehigh Univ., Bethlehem, Pa.

[17a] Received by the Secretary April 6, 1933.

explained the reason for the difficulty in selecting the proportional limit of the columns. A full report on tests of columns with known eccentricities referred to by them would be valuable.

It was hoped that further tests could be made on both riveted and welded columns of larger size and with variations in the welding and no doubt would have been under way at this time, as they had been planned before Professor Slater's death. It is possible that they may still be made, and the writer trusts that a number of the points suggested by Mr. Walker and others may be taken up and considered at that time.

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DISCUSSIONS

TESTS FOR HYDRAULIC-FILL DAMS

Discussion

BY JOEL B. COX, ASSOC. M. AM. SOC. C. E.

JOEL B. COX,[33] ASSOC. M. AM. SOC. C. E. (by letter).[33a]—The author has done a most praiseworthy piece of work in presenting the data derived from the testing program of the Cobble Mountain Dam to the profession. The time is ripe for a thorough-going consideration of the principles of hydraulic-fill dam construction, and of the possibilities of scientific design based on known physical facts. The establishment of adequate laboratory procedure is essential to this undertaking. The writer presents the following discussion in order that engineers may also have available for comparison the results obtained from a testing program at an entirely different dam, dealing with a material almost diametrically opposite in its physical characteristics from those met with at Cobble Mountain. In its scope and intent this program was considerably more extensive than that at Cobble Mountain, or than has heretofore been attempted in connection with hydraulic-fill construction. Following the precedent of the author's paper, a full report on methods used and results obtained, by Maitland C. Dease, Testing Engineer, McBryde Sugar Company, Limited, has been filed for reference with the complete manuscript of the author's paper in Engineering Societies Library.

The testing program at the Alexander Dam, Wahiawa Valley, Island of Kauai, Hawaiian Islands, completed in 1932, was developed in the belief that recent advances in the science of soil mechanics, and most notably, the work of Charles Terzaghi, M. Am. Soc. C. E., were sufficient to allow a complete analysis of the stresses within a hydraulic-fill structure and to enable design and control of construction to be based on such an analysis.

Professor Terzaghi's work[34] forms the starting point of the theoretical developments obtained in the technical control of the Alexander Dam. Early

NOTE.—The paper by Harry H. Hatch, M. Am. Soc. C. E., was presented at the Joint Meeting of the Irrigation and Power Divisions, Yellowstone National Park, Wyoming, July 7, 1932, and published in October, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1933, by Charles H. Paul, M. Am. Soc. C. E.; February, 1933, by Joel D. Justin, M. Am. Soc. C. E.; March, 1933, by Messrs. Jeptha A. Wade, and Stanley M. Dore; and April, 1933, by Messrs. D. P. Krynine, and M. M. O'Shaughnessy.

[33] Engr., McBryde Sugar Co., Ltd., Eleele, Kauai, Hawaii.

[33a] Received by the Secretary January 27, 1933.

[34] "Erdbaumechanik auf bodenphysikalisher Grundlage," by Charles Terzaghi, M. Am. Soc. C. E., Leipzig und Wien, 1925.

in the investigation it was found that tests did not always follow closely the rules laid down by Professor Terzaghi. The first addition to Professor Terzaghi's theory necessary to account for the observed facts was to assume that there were inter-particle forces of attraction even if the material was saturated and immersed in water.

Shearing resistance was, therefore, a compound phenomenon, being partly true cohesion, due to these little understood particle attractions and partly apparent cohesion and friction, a frictional effect caused by external load. This external load, in turn, could consist of a mechanically applied load or of a load applied by the surface tension of the water films at an external air boundary.

This recognition of true cohesion has also been made under Professor Terzaghi both by the U. S. Bureau of Public Roads[35] and at the Massachusetts Institute of Technology.

Later, it has been shown that the grains of soil cannot be considered as perfectly elastic bodies in the compression phenomena of a clay-like soil, but that their deformation is governed partly by complex frictional effects and partly by laws of plastic flow.

Other phenomena of great importance in the actual behavior of soils under stress arise from the inclusion of air in various states in the mass and in the effects of base replacement and colloidal effects due to absorption and the change in physical character coincident with a change in the acidity of the soil.

In many such relationships, it is apparent that the mineralogy of the soil, the actual chemical constitution of the various individual grains, is of the utmost importance. As yet no completely satisfactory methods have been devised for the identification of, or for experimenting with, such minerals, and much progress in the understanding of soil phenomena may be expected along these lines. No chemical analysis of a soil as a mass can be of final value without knowing how the elements are distributed in the various grains of different sizes.

In all this work, it has been apparent that great advantage to all concerned will result from a closer knitting together of the work of those who are studying soils from widely different angles. The agricultural soil physicist, the engineer concerned with soil, and the soil mineralogist and chemist can each learn much regarding his own problems by a closer contact with the work of all the others.

Specific Gravity Determination.—The pycnometer method adopted at the Alexander Dam consisted of drying the specimen 24 hours at 105° C in a pycnometer flask, adding distilled water and boiling it for 8 hours to eliminate dissolved, entrapped, and absorbed air. By using an excellent chemical balance, carefully tared pycnometers, accurate thermometers, and care in handling, results accurate to four figures can be obtained. Simple displacement methods were frequently in error as much as 15 per cent. As all subsequently determined constants are correlated with voids-ratio, an accurate

[35] *Public Roads*, Vol. XII, pp. 89 and 117.

knowledge of specific gravity is most important, and engineers should be warned of the difficulties and dangers of approximate methods.

Permeability.—Capillary flow methods of determining permeability proved unsatisfactory at the Alexander Dam as well as at Cobble Mountain. As is apparent from the author's Fig. 15 and all other work with the consolidation of clay-like materials, the simultaneous determination of voids-ratio and permeability is necessary. No permeability determination for which the corresponding voids-ratio of the material is unknown, is of value. The criticism of the large percolator shown in the author's Fig. 14, is that the exact determination of voids-ratio is difficult, if not impossible, and, in fact, it will not be uniform throughout the depth of sample. The criticism of the stand-pipe permeameter (the author's Fig. 13) is that it is difficult to load with stiff and well compacted material, at the same time keeping the material homogeneous throughout. A single loading gives a determination at one voids-ratio only.

Both these difficulties are eliminated by determining the permeability in the machine devised by Professor Terzaghi on a sample for which the voids-ratio is known accurately at all times.

The clay-like laterites involved in this work may be described as soft, usually reddish, residual decomposition products of basaltic lavas or pyroclastics. They have a low percentage of silica (about 22), and are correspondingly high in iron and alumina. The specific gravity of the soil particles averages 3.2, but for individual particles the value varies widely.

Such soils are characterized by exceptionally high voids-ratios. The volume of water in such a soil is usually twice that of the solid phase even when compacted under heavy pressure. No complete explanation of this phenomenon, and of the wide range and erratic variation of this feature within a group of apparently similar soils, is available. A part of such variation is caused by the past history of the soil relating to its moisture and pressure; and compacting under pressure is shown to be a slow process which will continue to a surprising extent long after existing theories would predict a state of equilibrium. The relationship between permeability and voids-ratio is not even approximately that given by Professor Terzaghi's theory,[36] but is such that it can often be represented by the formula:

$$\log_e K_0 = c_5 + c_6 E \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (45)$$

in which, K_0 is the reduced coefficient of permeability $= \dfrac{K}{1 + E}$; K, the coefficient of permeability; E, the voids-ratio, or ratio of voids to volume of solid material; and, c_5 and c_6 are constants.

When plotting $\log K_0$ against E on semi-logarithmic paper, tests on many soils will plot as straight lines, while others are concave upward and still others, concave downward. The relationship between pressure and voids-ratio follows very closely Professor Terzaghi's equations, thus,

$$E = - a_1 \log_e (p + p_c) - a_2 (p + p_c) + c \quad \ldots\ldots\ldots\ldots (46)$$

[36] "Erdbaumechanik," pp. 120 *et seq.*

in which, a_1, a_2, p_c and c are constants, and p is the unit pressure on clay, or, in approximate form,

$$E = - a_1 \log_e p + c_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(47)}$$

Usually, Equation (47) is a workable and closely approximate expression for soils.

It is interesting to compare Equations (46) and (47) with the relationships found by the author at Cobble Mountain. Equations (21) and (24) may be written, using the present nomenclature:

$$\log_e (1 + E) K_0 = c_7 + c_8 \log_e \frac{E}{1 + E} \ldots\ldots\ldots\ldots \text{(48)}$$

and,

$$\log_e \frac{1}{1 + E} = c_2 + 0.0428 \log_e p \ldots\ldots\ldots\ldots\ldots\ldots \text{(49)}$$

It would be interesting to try both sets of formulas on the original data to see which actually produces the better fit. There is little actual difference over a moderate range in voids-ratio.

It is in the investigation of chemical content, or mineralogy of soil grains. that methods and knowledge are altogether inadequate. The Hawaiian soil consists of an enormous variety of separate minerals. The specific gravity of the separate grains of a representative sample varied from 5.2 to 2.2. The identifiable grains range from magnetite to montmorillonite. It is clearly impossible to obtain much of value from the usual over-all or average determination of the chemical composition of such a soil. The most hopeful avenues of attack are to determine optically all those translucent particles for which such methods are applicable and to sort out the remainder by the use of the Thoulet solution (a mixture of solution of mercuric iodide and potassium iodide that can be given any specific gravity not exceeding 3.2) into convenient separates having approximately the same specific gravity. It is probable that further study of these separates will require the use of microchemical analytic methods.

Much of interest and importance both theoretically and practically was learned by careful microscopic examination of the soil. Many of the physical qualities most important from the standpoint of the engineer, such as crushing strength, flocculation or dispersion, and permeability can be estimated by eye by comparison with similar solids the characteristics of which are known.

A method of mechanical analysis or grading by size was developed using the direct measurement of particle diameters under the microscope. A comparison of the results of such determinations using sedimentation methods is most interesting. In every case (as is to be expected) the direct measurement gives larger grains than is computed from the sedimentation data, but the extent of the difference is very revealing of the character of the soil (for example, see Fig. 21).

The differences between the two methods of determining particle size arise from the following causes:

(1).—Visible Diameter Against the Diameter of the Equivalent Sphere.—The microscopic method measures not more than two diameters, which (due to the tendency of flat grains to orient themselves) will usually be the two greatest diameters of the grains. The sedimentation method determines the

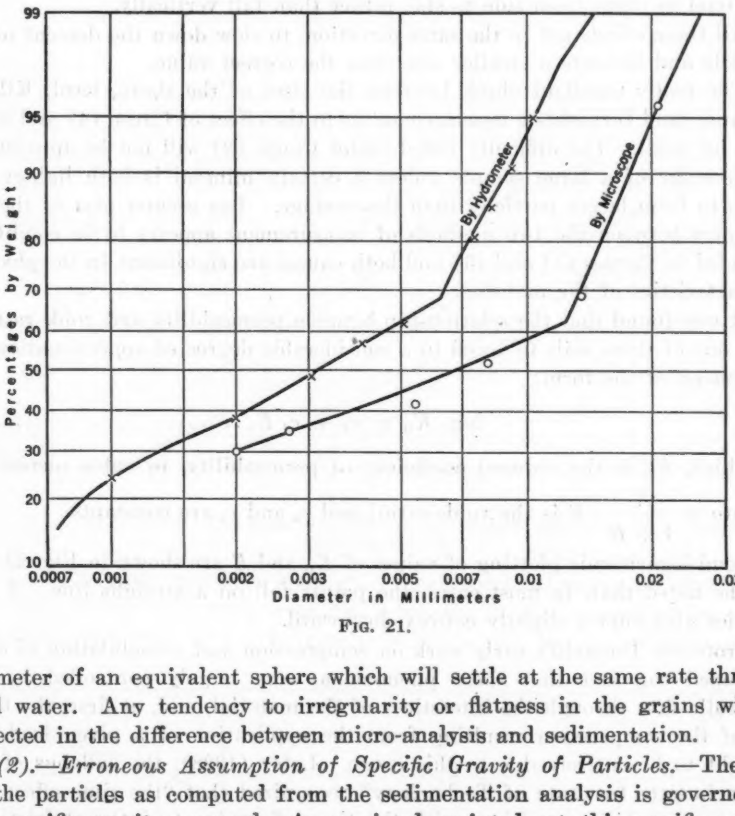

FIG. 21.

diameter of an equivalent sphere which will settle at the same rate through still water. Any tendency to irregularity or flatness in the grains will be reflected in the difference between micro-analysis and sedimentation.

(2).—Erroneous Assumption of Specific Gravity of Particles.—The size of the particles as computed from the sedimentation analysis is governed by the specific gravity assumed. As previously pointed out, this specific gravity may differ greatly in various grains, and, therefore, any individual grain (the specific gravity of which is unknown) may differ greatly in size from that computed by sedimentation analysis, using the average specific gravity of the material.

(3).—Expanded Particle.—The excessive voids-ratio of such soils as have been studied may be due in part to excessively hydrated or expanded particles, the water of expansion of which is driven off at a low temperature and, hence, does not appear in the specific gravity determination at 110° C. Such particles will measure much larger than their speed of descent indicates.

(4).—Failure of Experimental Conditions to Fulfill Assumptions of Stoke's Law.—The sizes in sedimentation analysis are computed on the assumption that Stoke's law for the velocity of descent through still water is strictly

fulfilled. It has been demonstrated[37] that this is not strictly true for soil particles. Mechanical and electrical interactions between the particles themselves and the sides of the container give non-vertical paths, while there is a certain amount of interference from the hydrometer bulb. Slight differences of temperature set up convection currents in the liquid, and flat particles will tend to plane from side to side rather than fall vertically.

All these effects act in the same direction, to slow down the descent of the particle and indicate a smaller size than the correct value.

The really excellent check between the sizes of the sharp, hard, Kilauea volcanic sand furnishes a measurement as to the effect of Cause (4) and shows it to be small. The difficulty listed under Cause (2) will not be apparent in the average of a large sample unless a certain mineral is both lighter and tends to form larger particles than the average. The greater part of the discrepancy between the two methods of measurement appears to be conditions included in Causes (1) and (3) and both causes are significant in the physical characteristics of the material.

It was found that the relationship between permeability and voids-ratio of each one of these soils followed to a considerable degree of approximation, an expression of the form:

$$\log_e K_0 = c_5 + c_6 E \qquad (50)$$

in which, K_0 is the reduced coefficient of permeability, in cubic meters per minute $= \dfrac{K}{1 + E}$; E is the voids-ratio; and c_5 and c_6 are constants.

Semi-logarithmic plotting of values of K_0 and E are shown in Fig. 22. It will be noted that, in most cases, the points fall on a straight line. A few samples give curves slightly convex downward.

Professor Terzaghi's early work on compression and consolidation of clays developed the fact that these phenomena were largely controlled by the hydraulic flow through the interstices of the material, and, at first, the time-lag of the compression resulting from the application of a given load was thought to be entirely due to this cause. Later (1928), the syllabus of the Massachusetts Institute of Technology[38] recognized that "the time effect due to low permeability combines with the time effect due to internal friction." By noting that the permeability effect varies with the square of the reduced thickness of the layer of soil, while the time effect due to internal friction is assumed to be independent of this thickness, a practical method of disentangling the two is developed. The research work at Alexander Dam, through two long-continued tests, gives additional information on this problem and suggests that the phenomenon is more complex than at first appears. One test compared the compression resulting from repeated applications of the maximum load and the rebound curves after the removal of this load. The resulting hysteresis loops are quite similar, but show a steady reduction

[37] Experiments by F. J. Viehmeyer, Prof. of Agri. Soil Physics, Univ. of California, Agri. School, Davis, Calif.

[38] Course in Soil Mechanics, Pt. I, "Soil Physics," by Charles Terzaghi, M. Am. Soc. C. E., 1928.

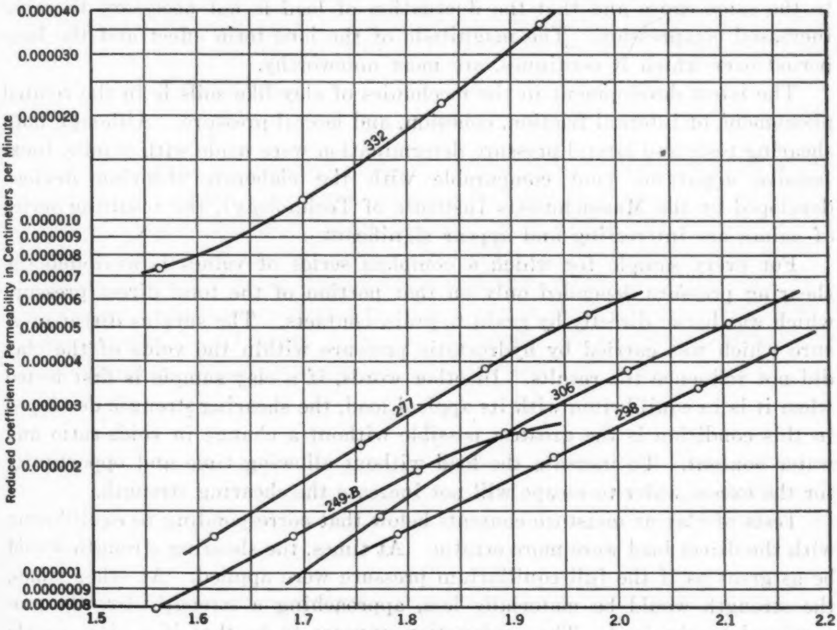

FIG. 22.—RELATIONSHIP BETWEEN VOIDS-RATIO AND PERMEABILITY OF CORE MATERIAL, ALEXANDER DAM.

of voids-ratio with successive loadings. The second test shows the slow steady reduction of the voids-ratio over many months of steady application of the same load (see Fig. 23). The results indicate that the two effects are due

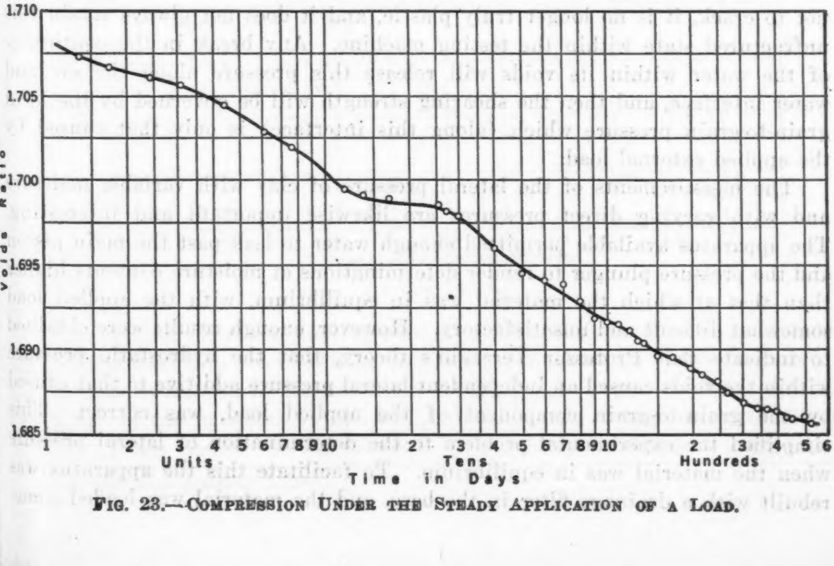

FIG. 23.—COMPRESSION UNDER THE STEADY APPLICATION OF A LOAD.

to the same cause and that the fluctuation of load is not necessary to cause increased compression. The magnitude of the long-term effect and the long period over which it continues, are most noteworthy.

The latest development in the mechanics of clay-like soils is in the related phenomena of internal friction, cohesion, and lateral pressure. Although both shearing tests and lateral pressure determination were made with simple, inexpensive apparatus (not comparable with the elaborate shearing devices developed by the Massachusetts Institute of Technology), the resulting series of values are interesting and appear significant.

For every sample for which a complete series of values is available the shearing pressure depended only on that portion of the total direct pressure which was borne directly by grain-to-grain contacts. The surplus direct pressure which was carried by hydrostatic pressure within the voids of the clay did not influence the results. In other words, if a clay sample is first tested when it is in equilibrium with its applied load, the shearing strength developed in this condition is the greatest possible without a change in voids-ratio and water content. To increase the load without allowing time and opportunity for the excess water to escape will not increase the shearing strength.

Tests of clay at moisture contents below that corresponding to equilibrium with the direct load were more erratic. At times, the shearing strength would be as great as if the full equilibrium pressure were applied. At other times, the strength would be materially less, approaching a strength directly proportional to the load. The explanation appears to be that if a clay sample contracts by drying without cracking, its interior is subjected to a pressure caused by the surface tension of the water between the grains great enough to cause it to be in elastic equilibrium; in other words, its voids-ratio is that corresponding to its actual internal pressure. Such a lump will have a shearing strength governed by this actual internal pressure, which may be many times that of the applied load. However, such a mass is too dry to be trusted not to crack, it is no longer truly plastic, and it does not always retain this unfractured state within the testing machine. Any break in the continuity of the water within its voids will release this pressure along the air and water interface, and then the shearing strength will be governed by the local grain-to grain pressure which (along this interface) is only that caused by the applied external load.

The measurements of the lateral pressure of clay with variable moisture and with varying direct pressures are likewise important and interesting. The apparatus available permitted enough water to leak past the main piston and the pressure plunger to render determinations at moisture contents higher than that at which the material was in equilibrium, with the applied load somewhat difficult and unsatisfactory. However, enough results were obtained to indicate that Professor Terzaghi's theory, that the hydrostatic pressure within the voids caused an independent lateral pressure additive to that caused by the grain-to-grain component of the applied load, was correct. This simplified the experimental problem to the determination of lateral pressure when the material was in equilibrium. To facilitate this the apparatus was rebuilt with a drainage filter in the base, and the material was loaded some-

what wetter than required and allowed to reach equilibrium in the machine. The results obtained showed a closely linear relationship between equilibrium or grain-to-grain pressure and the resulting lateral pressure. The total lateral pressure is, therefore,

$$L = Kp + w \dots\dots\dots\dots\dots\dots\dots\dots (51)$$

in which, L is the total lateral pressure; p, the grain-to-grain pressure within the soil mass; w, the hydrostatic pressure in water in voids of the soil mass; K the constant of lateral pressure; and $P = p + w$, the total pressure in the soil mass caused by applied load. The values of K obtained with soils at Alexander Dam, and using the apparatus available, ranged from 0.35 to 0.50.

A most important relationship with clays is that which occurs between shearing strength and lateral pressure. As shown[39] by Professor Terzaghi, the lateral pressure should be given by the following equation:

$$\frac{L}{P} = \tan^2\left(45° - \frac{\phi_o}{2}\right) \dots\dots\dots\dots\dots\dots (52)$$

in which, ϕ_o is the angle of internal friction, or $\tan \phi_o$ is the value of the shearing coefficient. The only complete series of tests on materials at Alexander Dam in which satisfactory data were obtainable on strictly duplicate and comparable samples both in shear and in lateral pressure, are given in Fig. 24.

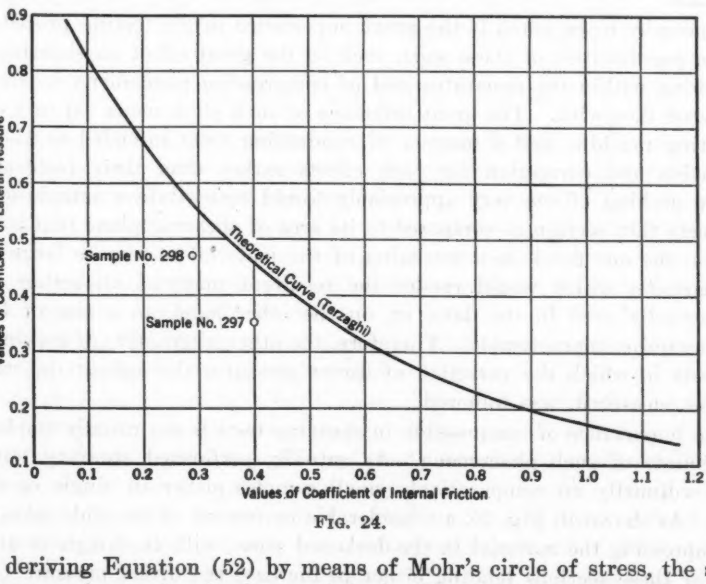

FIG. 24.

In deriving Equation (52) by means of Mohr's circle of stress, the shearing stress used must be closely scrutinized. The shearing stress developed to resist the formation of lateral pressure will be the true maximum shearing strength of the material. Not only is this the true maximum on any plane, but it is the passive or static stress. The shear measured in any test is

[39] "Erdbaumechanik," p. 188.

probably not quite this pure shear, but a complex phenomenon with changing areas and distorted lines of pressure. Besides, this difference between the true maximum shearing stress which governs the formation of lateral pressure and that shown in shearing tests, there is a possibility that the internal friction of a clay, like the friction between solid surfaces, has two values, one for static, and the other for moving or dynamic, conditions. If so, the movement of the slide allowed by pure compression of the material may be sufficient to prevent a true measurement of the static coefficient of shear.

The physical properties of soils characterized by a granular, or a crumb-like, structure are transitional between those of clays, or colloidal-cohesive soils, and clean sands. These properties are, therefore, more complex and more variable than those of either sands or clays, and, in general, the best understanding of such phenomena is obtained by a comparison between the observed action of the crumb-like soil and the corresponding performance expected from clay on the one hand and sand on the other. It is probable that soils may be found to vary by imperceptible degrees in a continuous series from the most perfect of clays to the most cohesionless sands. The contribution of any series of tests on such soils as those at the Alexander Dam is, therefore, more likely to be of practical rather than of theoretical value and to be restricted largely in its application to nearly identical materials.

Especially to be noted is the great importance in the testing procedure of certain peculiarities of these soils, such as the great effect on shearing tests of arching within the apparatus and of compression phenomena inextricably connected therewith. The great influence of such phenomena led to a design of testing machine and a manner of conducting tests intended to allow the estimation and correction for such effects rather than their removal. To remove arching effects very appreciably would necessitate a sample of such extremely thin section as compared to its area of shearing plane that it would lead, on the one hand, to a screening of the material to remove large lumps and particles which would render the resultant material altogether unlike that actually used in the dam, or, on the other hand, to a size of testing machine quite impracticable. Therefore, the other alternative of making control tests in which the variation of direct pressures throughout the mass of soil was measured, was followed.

The importance of compression in shearing tests is not usually emphasized in accounts of such phenomena. As actually performed shearing tests are made ordinarily on comparatively small samples either in single or double shear. As shown in Fig. 25, a considerable movement of the slide takes place by compressing the material in the darkened areas, with the toggle or arching effect of these sections tending either to increase the direct pressure (if the arrangement of slides is rigid), or to force back the material above or below (if a constant direct pressure is maintained). Before a true shearing failure takes place the shearing surface is considerably reduced in area and the distribution of stresses is complicated far beyond easy analysis. The shearing surface is no longer plane. Attempts to force shear to occur along a plane sur-

face by grids or other mechanical restraints instead of solving the problem, only complicate it further and introduce enormous uncertainties as to the distribution of the direct pressures. Much more elaborate and thoughtful experimentation is necessary before the measurement of shearing strength may be considered as satisfactorily obtainable.

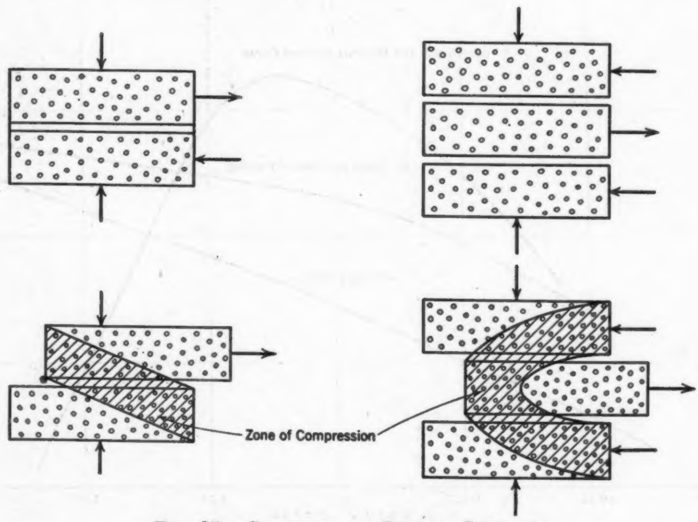

FIG. 25.—COMPRESSION DURING SHEARING.

An interesting series of tests on crumb-like soils was made to show the relationship between cohesion, moisture content, and time and pressure under which the material is packed and the cohesion developed. These are shown in Fig. 26. Cohesion was measured by means of shearing tests at light loads with an estimated correction for friction as determined from tests on the same material at varying pressures. The shearing force required, therefore, is extrapolated to zero direct pressure.

Referring to Fig. 26, the slow increase in cohesion with time after two or three days (Curve *B*), the rapid increase with pressure (Curve *C*), and the existence of moisture content both above and below which the cohesion is reduced (Curve *A*), are all clearly shown. These curves were determined by tests of the same sample under different conditions; for example, for Curve *A*, the sample was packed 2 days under a pressure of 4.97 kg per sq cm (71 lb per sq in.); for Curve *B*, the sample was kept under the same load (4.97 kg per sq cm) during the test, at a constant moisture content of 43%; and for Curve *C*, the sample was kept at a moisture content of 43% and after 6 hours was tested under various loads.

To correlate the results of compression tests in the large Terzaghi apparatus with settlements and changes in permeability observed in the dam, it is necessary to allow for the influence of larger lumps than can be tested within this apparatus. As an aid in such consideration a series of crushing

tests of 2-in. cubes cut from such lumps, was run under a variety of moisture conditions. In general, an optimum moisture content is not observable with these samples. Some of them showed such an increase in strength with moderate drying, followed by a decrease as drying continued, but most sam-

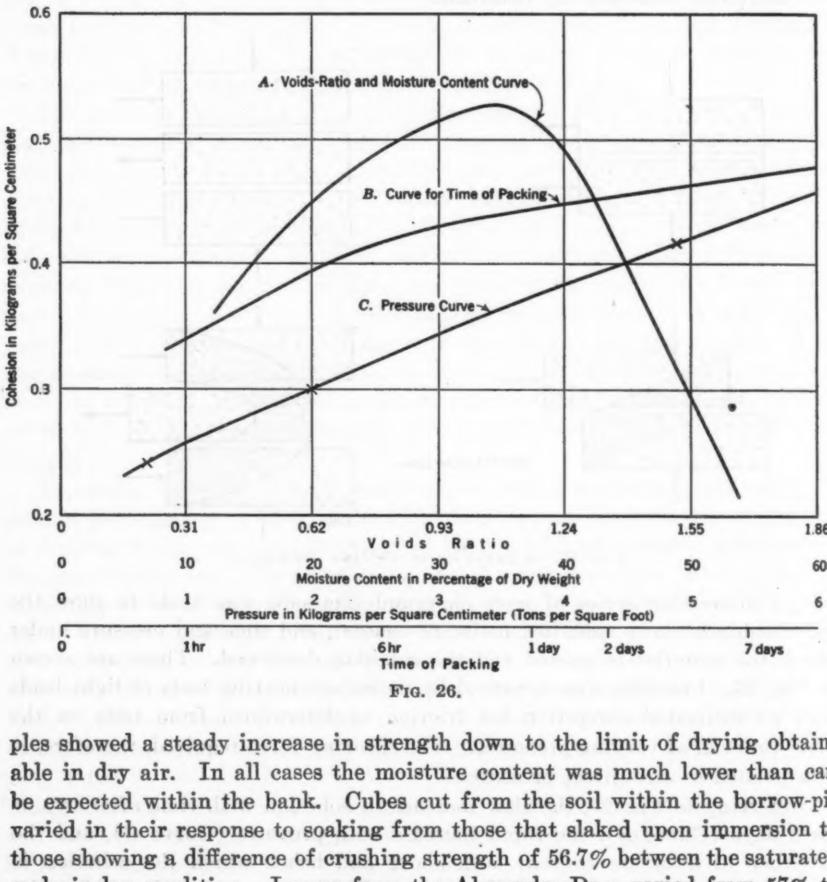

FIG. 26.

ples showed a steady increase in strength down to the limit of drying obtainable in dry air. In all cases the moisture content was much lower than can be expected within the bank. Cubes cut from the soil within the borrow-pit varied in their response to soaking from those that slaked upon immersion to those showing a difference of crushing strength of 56.7% between the saturated and air-dry condition. Lumps from the Alexander Dam varied from 57% to 86% reduction in strength. Random samples showed a non-systematic variation in strength from 20 lb per sq in. to 100 lb per sq in., when soaked.

It is apparent that the settlement and compacting of such a body of earth as the beach areas of this dam, takes place not only by the compression of the finer-grained material, but also by the crushing of the coarser lumps.

The consolidation formula proposed by the author (Equation (30)) for a dam subsequent to completion is defective in its derivation. Equation (29) is based on the statement that "the rate of consolidation of the material under a given constant load, shortly after the application of the load, becomes almost uniform." This unfortunately is far from the case. This problem has been

given its theoretical outline by Professor Terzaghi and has had two solutions proposed. One by Glennon Gilboy, Jun. Am. Soc. C. E.,[40] is exceedingly neat in its mathematical derivation. Its major defect is in neglecting the effect of gravity of the movement of the escaping water. The second method is one proposed by the writer for use in computing the consolidation of a dam core during construction.

It consists in separating the cross-section of the dam into a series of finite divisions and applying Darcy's law to each division separately for a small interval of time. Both divisions and time intervals must be chosen small enough to allow an average uniform velocity to be used without the introduction of too great errors.

A demonstration of the relative amount of the errors occasioned by the assumption of constant rate of consolidation is given by computing the rate of consolidation of the Cobble Mountain Dam by Professor Gilboy's method.

Taking an elliptical cylinder (semi-major axis, 5 930 cm, and semi-minor axis, 4 780 cm) that has an area roughly equivalent to that of the dam core, and a ratio of diameters about that of height to one-half the width of the actual core; and assuming an average percentage of voids of 42.8, with a Terzaghi coefficient of consolidation of 1.4, gives the following rate of consolidation:

Percentage of consolidation	Time, in days	Percentage of consolidation	Time, in days
0	0	60	501
10	21.8	70	705
20	62	80	1 010
30	134	90	1 550
40	226	95	2 085
50	365		

This is rapid consolidation, indeed, owing to the relatively coarse material of the core and the extremely narrow cross-section obtained. Even so, however, if it is estimated that 50% of the consolidation will occur during construction, the percentage of consolidation at various times after completion may be estimated, as follows:

Years after completion	Percentage of consolidation
0	50
1	71
2	82
3	88
4	93
5	96

The slowing down in rate of drainage is very marked, the drainage during the third year being only 29% of that during the first.

The writer takes exception to the argument presented under the heading "Consolidation of the Core of Cobble Mountain Dam," purporting to dis-

[40] "A Theoretical and Experimental Investigation of the Properties and Behavior of Hydraulic-Fill Dams," Thesis presented to Mass. Inst. Tech., in 1928.

credit the use of consolidation computations. Seven points are made, which will be taken up in order:

(1) There is no reason why loads cannot be applied in the laboratory in any way desired, and, hence, they can be arranged to duplicate conditions in the dam.

(2) The initial percentage of voids can likewise be made anything desired when conducting laboratory tests.

(3) Two questions are raised in this connection. The first is that laboratory tests thus far have been on small-sized test pieces. The largest in the writer's knowledge have been those in the large machine at the Alexander Dam, 8 in. in diameter. These samples checked admirably with similar ones tested in the smaller machines, provided consolidation controlled by hydraulic action is separated from that controlled by internal friction. The writer hopes that more work can soon be done on large-scale and long-time tests, which will give a final answer to the question. In the meantime, nothing has yet been shown sufficient to render computations based on Professor Terzaghi's hydraulic theories so seriously defective as to be valueless. The second point is the great variability to be found in small samples of apparently similar soil. This is a fact encountered throughout the realm of soil testing, and can only be met by such a number of individual tests that their average will represent fairly that of their large-scale prototype.

(4) The rough surfaces of the beaches go down with the core as it compresses and, hence, do not retard its consolidation. The effect of abutments is certainly to complicate the process in close proximity to them, but a computation near the center of the dam will usually be sufficient for engineering purposes.

(5) The time intervals for the hydraulically controlled portion of the consolidation may be exactly computed for strict similarity. Slow re-adjustment over a long time is shown by the Alexander Dam tests (Fig. 23). This argument is precisely similar to the first point in Argument (3).

(6) The drainage facilities are capable of investigation and allowance may be made for them.

(7) The variable pressures and stresses are susceptible to computation.

Table 5 presents in compact form the great contrast in physical properties presented by the core materials of Alexander Dam and Cobble Mountain, and may well explain why methods applicable to one type will not be usable with the other.

In conclusion, the writer wishes to emphasize four points, one of which is in direct accord with the author's views while three present modifications:

(*a*) Laboratory and field determinations of the physical constants of soils are already established on a sufficiently theoretical and practical basis to serve as useful tools for the designing engineer.

(*b*) Much further investigation is urgently needed in the fields of scale effect, shearing and lateral pressure phenomena, soil mineralogy, and other phases of soil physics and mechanics.

TABLE 5.—COMPARISON OF AVERAGE PHYSICAL CONSTANTS OF CORE MATERIAL

Description	Cobble Mountain Dam	Alexander Dam	Description	Cobble Mountain Dam	Alexander Dam
Effective size, in millimeters	0.009	0.00448	Constants for Change of Permeability:		
Uniformity coefficient.....	6.0	10.27			
Specific gravity..........	....	3.117	c_s, in Equation (45)....	3.93	4.07
Voids-Ratio; at Loads of:			m, in Equation (21)....	6.4	7.2
50 grams per sq cm.	1.05	2.25	Consolidation Coefficient, for $p = 3\,250$ Grams per Sq Cm:		
3 250 grams per sq cm.	0.72	1.682			
Compressibility coefficient, $\frac{1}{a_1}$..................	5.63	5.62	c (Terzaghi)..........	1.88	0.0451
Rebound coefficient, $\frac{1}{A}$.....	....	77.93	$C = [c(1+E)^2]$, for constant volumes.......	5.58	0.336
Permeability coefficient (in centimeters per minute, or meters per day) at a load of 3 250 grams per sq cm..................	0.000178	0.0000064	Lower liquid limit (percentages)............	....	69.6
			Plastic limit (percentage).	....	55.5
			Plasticity index (percentage).................	....	14.1
			Moisture equivalent.....	....	57.9
			Hydrogen-ion concentration, pH.............	....	5.9

(*c*) Although the writer appreciates the motive for the author's opinion that "it is time that a definite step is taken and a standard method developed in making tests for this particular kind of construction," he does not believe that any present method of conducting tests is of sufficiently proved superiority to be worthy of such elevation to the position of standard. The writer suggests that the most helpful course at present is to compare the constants obtained from each method of testing which is being followed, and to find the extent and reason for such differences as appear. It is the physical constants of the large-scale prototypes that are desired, and standard laboratory tests are of value only as they correlate with these values.

(*d*) All recent work on soil mechanics, using different types of soil, emphasizes the unreliability of depending on the size of grain, at least in the sizes determinable by sieve and sedimentation or elutriation analyses, as a criterion for estimating the physical constants of the soil as a whole. For most purposes direct laboratory determinations of the major constants of compressibility, elasticity, permeability, internal friction, and cohesion are necessary. One reason why the mechanical analysis of grain size is unsatisfactory as a criterion is that microscopic and elutriation methods cannot be carried into the range of colloidal grain sizes, and it is the colloidal content of many soils that governs a considerable number of its physical properties. Another reason is that mechanical analysis gives no insight into the realm of packing or structure, the importance of which is especially great in foundation problems, as pointed out by Casagrande.[41]

Finally, the thanks of the profession are due to the author for undertaking the labor of preparing the paper summarizing the testing experience at Cobble Mountain Dam. It is to be hoped that no major project of this kind in the future will be undertaken without adequate test data and the fullest possible use of such data in predicting and explaining the full-scale performance of the prototype.

[41] "Structure of Clay in Foundation Engineering," *Journal*, Boston Soc. of Civ. Engrs., April, 1932, pp. 168 *et seq.*

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

GEORGE WASHINGTON BRIDGE: ORGANIZATION, CONSTRUCTION PROCEDURE, AND CONTRACT PROVISIONS

Discussion

BY EDWARD W. STEARNS, M. AM. SOC. C. E.

EDWARD W. STEARNS,[8] M. AM. SOC. C. E. (by letter)[8a]—The writer appreciates greatly the interesting communication from Mr. Carlin, although he cannot agree with him in all his comments. Mr. Carlin's remarks have especial reference to the contract for the approach tunnel under 178th Street, Manhattan, where abutting properties were subject to damage by the conduct of the construction operations. The writer is entirely in accord with the importance of a preliminary examination of such properties and believes that, imperfect as such a proceeding is, it is, nevertheless, if properly done, the one best way to establish the extent of damages caused by the work. He believes, however, that inasmuch as such a determination is of first importance to the contractor, the undertaking of such an examination should originate with the contractor himself and not with the owner.

The advisability of including a provision in the contract for the approach tunnel under 178th Street requiring such preliminary examination was seriously considered, but such provision was believed to be superfluous in view of the clause in the contract that provides that "the contractor shall do all things necessary or proper for or incidental to the work," in addition to the provision that "the contractor shall be the insurer of the Port Authority against the work of damage to the property of third persons, arising out of or in connection with the performance of the work." If, as Mr. Carlin indicates by his comments, the contractor considered a preliminary examination necessary and proper, then, by the terms of the contract, such an examination was required as part of his contractual duties. The Port Authority holds that such an inspection is primarily in the interest of the contractor himself and if included as a requirement of the contract, would tend at least to make the Port Authority jointly responsible with the contractor.

NOTE.—The paper by Edward W. Stearns, M. Am. Soc. C. E., was published in October, 1932, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: December, 1932, by J. P. Carlin, M. Am. Soc. C. E.

[8] Asst. Chf. Engr., The Port of New York Authority, New York, N. Y.

[8a] Received by the Secretary April 21, 1933.

Mr. Carlin makes the general statement that "if the owner of an improvement is injured by its delay in completion, it follows that he must benefit if the work is completed earlier than the contract time." This statement would be quite axiomatic if it were not a fact that in most undertakings, and particularly so in public work, there are usually external conditions over which the owner has no control and which are likely to make it impossible to reap the benefit of earlier completion. The writer is very glad that Mr. Carlin agrees with him that a "penalty and bonus" provision in its contracts would not have benefited the Port Authority. The proper co-ordination of the various elements affecting the completion of an undertaking is a problem, the complexity of which varies more or less directly with the magnitude and complexity of the undertaking itself, and is just as much a real problem for the engineer to solve as are the problems of the locations of the piers or the sizes of the various members. An engineer is remiss in his duty if he fails to give due consideration to all these elements and then inserts a penalty and bonus provision in his contract to cover up his laxity. In the writer's opinion, it is far better to take account of the conditions which must be met, arrive at a reasonable and logical time for the work to be completed, and then draw a contract which will either lead to its completion within that time or reimburse the owner for any loss which he may suffer by reason of the lateness of its completion.

It would seem as if there were little difference in dollars and cents between the two systems under the competition system of bidding. If the time set up is ample, the contractor is likely, under the bonus system, to predetermine his probable bonus and then deduct that amount from his bid price, whereas under the system of liquidated damages he will estimate the most economical time for him to consume in doing the work, and his bid price will reflect the resultant economy. If, on the other hand, the time set up is very limited, so limited in fact, as to make it difficult or impossible to complete the work on time, the contractor will increase the amount of his bid price so as to be reimbursed either for too rapid work or for the penalty imposed for failure to complete on time, or for both.

In closing, the writer wishes to reiterate both for himself and the staff of the Port Authority, his sincere appreciation and regard, not only for the whole-hearted co-operation from all the contractors on this great work, but also for the careful study given the problems facing them and the intelligence with which these problems were solved.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

A HISTORY OF THE DEVELOPMENT OF WOODEN BRIDGES

Discussion

BY MESSRS. H. FUGL MEYER, W. E. HOWLAND, HARRY K. ELLIS, AND NELSON J. BELL.

H. FUGL MEYER,[85] ASSOC. M. AM. SOC. C. E. (by letter).[85a]—A brief statement as to the contribution of the Chinese to the art of bridge building is of importance in establishing the origins discussed in this interesting paper. The Chinese character, "chiao," denoting a bridge, contains in its components the idea of a lofty wooden construction. The character was formed about 1003 B. C., thus lending authority to legendary history, which records the information that the Chinese had undertaken the construction of fairly large wooden bridges at a very early date. From the Confucian annals and from the history of the Near East it is known that in a number of Asiatic countries wooden pontoon bridges of considerable length were in regular use before 600 B. C.

At that early date there were a number of communities with a fairly high civilization in the part of Central Asia that now (1933) forms the western border of China. Because the turbulent and wide mountain streams in this territory made all inter-communication difficult, Nature made the inhabitants bridge builders long before written records of their kingdoms begin.

A number of interesting wooden cantilever and suspension bridges are still to be found in these regions. Whenever the width of the stream is less than, say, 130 ft, a cantilever appears to be preferred. Suspension bridges of plaited bamboo ropes are used for spanning streams as wide as 250 ft.

Multiple-span bridges of both types of construction are still extant, the longest being a suspension bridge of 700 ft.

NOTE.—The paper by Robert Fletcher and J. P. Snow, Members, Am. Soc. C. E., was published in November, 1932 *Proceedings*. Discussion of this paper has appeared in *Proceedings*, as follows: January, 1933, by Henry B. Seaman, M. Am. Soc. C. E.; February, 1933, by Messrs. Jasper O. Draffin, E. K. Morse, and John W. Storrs; March, 1933, by Messrs. C. J. Hogue, William A. Oliver, Philip G. Laurson, Richard S. Kirby, Wells N. Thompson, Benjamin Wilder Guppy, Ivan E. Houk, and William H. Baker; and April, 1933, by Messrs. Oren Reed, R. P. Davis and L. L. Jemison, Peter T. Landsem, J. K. Finch, A. A. Eremin, Aksel Andersen, and William G. Atwood.

[85] Mgr., Free Port of Copenhagen, Copenhagen, Denmark.

[85a] Received by the Secretary April 14, 1933.

Chinese bridges are generally the principal gathering place for the inhabitants of the neighboring town, and, therefore, they are often covered with a roof or are fitted with supports for a light roof of bamboo matting as a protection against sun and rain. Fig. 45 represents a bridge in the small

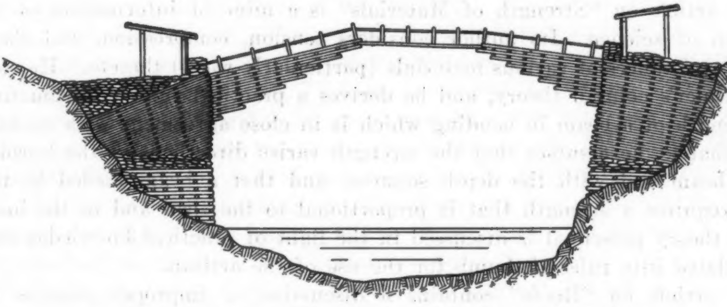

FIG. 45.—CANTILEVER BRIDGE IN THE KINGDOM OF MULI.

Tibetan-Chinese Kingdom of Muli. It will be noted that two heavy stone-filled wooden caissons lend stability to the cantilevers. The central part of the span can be removed when the King of Muli finds it necessary to block this entrance to his realm.

W. E. HOWLAND,[86] ASSOC. M. AM. SOC. C. E. (by letter).[86a]—In an "Encyclopædia or Dictionary of Arts, Sciences and Miscellaneous Literature," printed in Philadelphia, Pa., for Thomas Dobson, in 1798, and in the "Supplement" published in 1803, several articles appeared relating to wooden bridges, namely, "Arches," "Centering," "Carpentry," "Roofs," and "Strength of Materials," all related one to another with numerous cross-references. This twenty-one volume compilation[87] offers an opportunity for the student of the history of the development of technology to determine what was known about a particular science at a given period, and, in this case, it shows what knowledge might have been carried over from Europe to America. Such a medium as this could have brought to the early American bridge builders the knowledge gained by their European colleagues, not only in building wooden bridges, but also in the science of the equilibrium polygon and in countless practical details of wood construction for carrying heavy loads.

Although, as the authors have pointed out, the European wooden bridges were not especially noteworthy at this time, certain other Continental wooden construction appears to have been developed to a high degree of perfection before the era of American wooden bridge construction, for example, the wooden centering of Perronet and the wooden roof trusses of Sir Christopher Wren. The writers of this encyclopædia assembled this knowledge and made it available in a convincing and understandable form for the potential bridge

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[86a] Received by the Secretary April 1, 1933.

[87] Reprinted from "Encyclopaedia Britannica," or a Dictionary of Arts, Sciences and Miscellaneous Literature," Third Edition, printed for A. Bell and C. Macfarquhar, Edinburgh, Scotland, 1797.

builder. It seems inconceivable that a reader of these articles would not have profited immensely both in obtaining a clearer mental picture of the principles involved and the manner in which the forces distribute themselves in a complicated frame, but also in adding to his knowledge of practical details of construction.

The article on "Strength of Materials" is a mine of information to the historian of science. Its author considers tension, compression, and shear and the resistances of various materials (particularly wood) thereto. He considers Euler's column theory, and he derives a practical rule for estimating the strength of a beam in bending which is in close agreement with modern ideas; that is, he assumes that the strength varies directly with the breadth of the beam and with the depth squared, and that a beam loaded in the center requires a strength that is proportional to the span and to the load. All the theory presented is discussed in the light of practical knowledge and is translated into rules-of-thumb for the use of the artisan.

The article on "Roofs" contains a discussion of improper practice in joining wooden members, and it ends with the maxim of Perronet, namely, "to make all the shoulders of abutting pieces in the form of an arc of a circle having the opposite end of the piece for its center."[88] Then, to connect this subject of roofs with other phases of wood construction, the author adds this sentence,[89] "much of what has been said on this subject may be applied to the construction of wooden bridges and the centers for turning the arches of stone bridges."

The work of Perronet is discussed in the article on "Centering,"[90] with critical insight and yet with admiration. This section treats of the history of the development of the center to produce the stiffness needed for maintaining the shape of the stone arch in various stages of its construction. The author commends the center used by Mr. Robert Mylne (1734-1811) for the Blackfriars Bridge, at London, England. This bears a striking resemblance to a Bollman truss, with the compression members and tension members reversed. The construction consists of a number of pairs of struts, each one beginning at an abutment and terminating at a joint on the outside of the centering, the entire framework being joined together to form a structure of great rigidity.

In the article on "Carpentry,"[91] the author again refers to the work of Perronet. He states, for example, that:

> "The strongest of all methods of piecing a tie-beam would be to set the parts end to end and grasp them between other pieces on either side * * *. Mr. Perronet used it for the tie-beams or stretchers by which he connected the opposite feet of a center which was yielding to its load and had pushed aside one of the piers above 4 in."

The author then describes the use of scarfing employed by Perronet for eliminating bending in the bolts that connect wooden members. Two full

[88] "Encyclopædia or Dictionary of Arts, Sciences, and Miscellaneous Literature," 1798, Vol. 16. p. 476.

[89] *Loc. cit.*, p. 480.

[90] *Loc. cit.*, Supplement, 1803, Vol. 3, p. 234.

[91] *Loc. cit.*, p. 212.

pages of these and other joints are included, and two pages of illustrations of roof trusses, one with a clear span of 80 ft, and a sketch of the walking beam truss used by Watt and Boulton on their steam engine. It has tension members for the top chord.

The author on "Centering" discusses disapprovingly the wooden arch built by Grubenhamm, at Wittingten, Switzerland.[92] He then refers with admiration to a "bridge over the Portsmouth river in North America, * * * more than 250 ft in length * * * of several parallel arches of beams." The inventor said to be a Mr. Blodget, is quoted as saying that he found the strength of this bridge so great that he could with perfect confidence make one of four times the span. Could this be the Piscataqua Bridge of Timothy Parker built in 1794 and referred to by Messrs. Fletcher and Snow?

Finally, in recommending the best form for a wooden bridge, the contributor of this article returns to the ideas of the roof truss and the arch centers, concluding with these significant words:[93]

"The reader must perceive that we have now terminated in the construction of the Norman roof. We indeed think it the best general form, when some moderate declivity is not an insuperable objection. When this is the case, we recommend the general plan of the centering of the bridge of Orleans. We would make the bridge (we speak of a great bridge) consist of four trusses; two to serve as the outsides of the bridge, and two inner trusses, separating the carriage-way from the foot-paths. The road should follow the course of the lower polygon, and the main truss should form the rails. It might look strange; but we are here speaking of strength; and evident, but not unwieldly, strength, when once it becomes familiar, is the surest source of beauty in all works of this kind."

Harry K. Ellis,[94] Esq. (by letter).[94a]—The authors state that "as late as 1930, there were between 450 and 500 covered bridges in use." This must be an under-estimate, because in Chester County, Pennsylvania, alone there are forty-four covered wooden bridges at the present time. Nine were torn down in 1932, so that in this County there must have been at least fifty-three covered bridges in 1930. Many of the other counties in Eastern Pennsylvania still have large numbers of such bridges.

Among the bridges still existing in Chester County is one of 100 ft span, built in 1807 and still carrying traffic, and one of 60 ft span, built in 1819. The latter bridge is shored up at the present time. The writer believes that the one built in 1807 must hold the record for long life of Burr truss bridges, since Burr's first bridge was built only three years earlier.

Most of the covered bridges in Chester County are of the Burr type with the wooden arches extending to a height about 2 ft below the eaves. White pine was used in almost every case. Some of the bridges are of the queen-post type, without diagonals in the middle panel, and with the floor-beams hung upon a single bolt which passed through the center of the floor-beam upward through the lower chord and about 1 ft to 1.5 ft higher through the

[92] "Encyclopædia or Dictionary of Arts, Sciences, and Miscellaneous Literature: Supplement," 1803, Vol. 3, p. 248.

[93] *Loc. cit.*, p. 249.

[94] County Engr., Chester County, West Chester, Pa.

[94a] Received by the Secretary April 4, 1933.

truss post. A notch cut in the post permitted the insertion of a nut into which the bolt was screwed. Usually, this notch was on the outside of the truss and covered with the siding boards so that inspection was difficult.

It is true that the floors were the weakest part of these bridges, but the writer has twice seen a perfect compression failure of the arch rings. In both cases the wood broomed out on all four sides of the ring at the point of failure. In both cases, the arch was subject to a greatly increased load due to the bridge as a whole being much out of line.

Nelson J. Bell,[95] M. Am. Soc. C. E. (by letter).[95a]—This paper is most instructive, especially since technical literature in the past half century seems to have almost wholly neglected the subject. The general opinion among engineers has come to be that the old wooden highway bridges are fit objects of sentimental interest, but of very little practical worth in this age of steel and concrete. The paper by Messrs. Fletcher and Snow should bring home the fact that with only nominal care these old bridges have served their purpose with almost 100% efficiency over periods of 50 to 100 years, and, therefore, may be worthy of close technical study before it is decided whether one should be further preserved for practical use or should be condemned and scrapped. The mortality of these old bridges has been greatly increased in recent years because of the lack of knowledge concerning them and the consequent offhand opinion that they are unsafe under present-day traffic.

The preponderance of evidence brought out in this paper is to the effect that the bridges built by Wernwag, Burr, and most of their imitators, had an excess of strength everywhere except in their floor systems and lateral bracing, but no one has as yet described a method by which the distribution of stresses in the trusses may be analyzed. An exact analysis is probably impossible, and, in any case, would involve so many unknown factors that it would be highly impractical. In some cases it is possible, however, to arrive at a reasonable analysis of the primary stresses in the members by means of assumptions and simple graphic statics.

Several years ago, the writer was employed to investigate a wooden bridge spanning the Kentucky River at Camp Nelson, about twenty miles south of Lexington, Ky. This bridge was built in 1838 by Louis Wernwag. It has a single span of 240 ft and is composed of three trusses which are identical in all respects, except that the outer ones are curved horizontally so as to form flat arches, equal and opposite to each other. The outer trusses are spaced about 25 ft, center to center, at the middle of the span, and about 30 ft, center to center, at the ends. The center truss is straight and bisects the bridge, forming two one-way roadways. The general design is so similar to that of the bridge over Cheat River, Preston County, West Virginia (Fig. 35), that both must have been designed by the same man. The Camp Nelson Bridge is much heavier, the arches being composed of six pieces, in two rings of three pieces each. There is also a double web system, the main diagonals being composed of two pieces each, passing outside the arch rings, and the single-piece

[95] Cons. Bldg. Engr., Dayton, Ohio.

[95a] Received by the Secretary April 24, 1933.

counters passing between the two arch rings. The verticals have three pieces each, the center one passing between the arch rings and the others passing outside. Thus, the arches are clamped against lateral deflection by the verticals and diagonals. The verticals are dapped over the arches and bolted to them, but the diagonals have no connection to the arches, except by friction. An illustrated description of this bridge, with a tabulation of maximum stresses and an outline of the method of analysis was published[96] in 1928 by the writer and J. K. Grannis, M. Am. Soc. C. E.

It seems to be characteristic of Wernwag's bridges that they have heavy arches with comparatively light stiffening trusses, and that their ends are anchored to the abutments by tension rods. The Camp Nelson Bridge is composed essentially of three arches, each braced laterally and vertically by a stiffening truss, and the whole tied together with lateral trusses at top and bottom. The points of application of loading on each arch are spaced uniformly along the perimeter of the arch by means of the inclined posts, set normal to the curve of the arch. Dead and uniform live loading, therefore, causes compression in the arch, coincident with its center line. The truss is not stressed except by eccentric loads, under which condition it distributes the loads uniformly to the arch, thus limiting all stress in the arch to compression. The stress distribution can be reasonably compared to that in a suspension bridge, the only difference being that in this case there is an arch in compression instead of a cable in tension. With any eccentric load on the truss, it tends to rotate on the arch. This must be counteracted either by the dead weight of the truss, or by anchoring the ends to the abutments, and it is interesting to note that the trusses of the Camp Nelson Bridge were actually anchored by wrought-iron rods extending from the tops of the end posts down into the masonry.

The trusses by Burr and some others do not lend themselves to stress analysis as readily as those by Wernwag, as they seem to have been designed on the principle of a simple truss aided and strengthened by arches, so that there is no definite place where truss action stops and arch action begins. It would certainly be on the safe side to make the analysis on the basis of a stiffened arch where the arches have adequate lateral and vertical support to prevent distortion, but as the dead loads of these bridges greatly exceed the live loads, it is more practical to assign the dead load to the arches and the live load to the trusses and analyze them separately. In any case, the judgment of the investigator enters largely into the problem, and must supplement the mathematical work.

The main point which the writer wishes to make is that the old wooden bridges should not be condemned and scrapped just because they were not designed with the aid of present-day engineering knowledge and standards. Many of them can be made safe and serviceable for modern traffic by very simple repairs and reinforcement of their floors and lateral bracing systems, at a small fraction of the cost of removal and replacement.

[96] *Engineering News-Record*, Vol. 100, No. 6, February 9, 1928, p. 234.

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DISCUSSIONS

A PROBLEM OF SOIL IN TRANSPORTATION IN THE COLORADO RIVER

Discussion

BY MESSRS. WILLIAM T. COLLINGS, JR., R. F. WALTER, HARRY F. BLANEY, AND CALVIN V. DAVIS AND DIRK A. DEDEL

WILLIAM T. COLLINGS, JR.,[39] M. AM. SOC. C. E. (by letter).[39a]—Interesting data on the problem of soil transportation have been presented in this paper. The author has provided a conception of the magnitude of the enormous quantity of material carried by the Colorado River, both in suspension and along its bed. This problem is one that has confronted engineers on irrigation projects in the Lower Colorado delta region for years, and much has been written on the subject. As the author suggests, a thorough understanding of these data is necessary by those engaged in the design of irrigation works on, or leading from, the river.

The writer is particularly interested in a consideration of the merits of a plan proposed by the author for eliminating the desilting basins shown on the Government design by substituting a horizontal partition separating the flow to the head-gate structure. The author makes the following statement:

"A diversion structure placed across the river normal to the stream, provided with such a partition and under-sluices, properly proportioned, will exclude, automatically, all the bed load and that part of the suspended load in the lower water that passes under the partition."

A number of questions arise as to whether this statement can be borne out in entirety in application to the head-works of the All-American Canal, considering factors of major importance, such as magnitude of the structure, fluctuating river flow, available sluicing heads over and above irrigation requirements on the canal at periods of low-river discharges, shoaling, dependability of operation, maintenance, etc.

Under normal or average conditions of river inflow to Boulder Canyon Reservoir immediately after the completion of Imperial Dam—and for the first few years of the operation of the head-works, and when power demands

NOTE.—The paper by S. L. Rothery, M. Am. Soc. C. E., was published in December, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1933, by Messrs. C. E. Grunsky, Ivan E. Houk, and H. M. Rouse.

[39] Engr., Imperial Irrig. Dist., Imperial, Calif.

[39a] Received by the Secretary March 10, 1933.

both on Hoover Dam and on the Canal System have not been fully developed —there should be no occasion for close conservation of sluicing water at the head-works. However, firm power from Hoover Dam has been based upon output obtainable in low run-off periods as determined by run-off records from the Colorado River over a period of thirty years or more. The same conservative principle should be applied in the design of desilting works as regards capacity for sluicing operations in anticipation of possible low-period discharges. The possible demand for diversion at Imperial Dam as of date of completion will be close to 12 000 sec-ft, taking into consideration water to the Yuma Project, present Imperial and Coachella irrigation demands, and power installation. Assuming a maximum of 3 000 sec-ft, or the normal head-gate capacity of one of the desilting basins as sluicing head, the river flow required at the head-works would be 15 000 sec-ft. Under full development of the canal project, the full capacity of 15 000 sec-ft for the first reach of the canal to the drop for the Yuma Project is anticipated. Sluicing head then becomes a quantity over and above 15 000 sec-ft. It can be seen that under conditions to be anticipated during periods of low-river flow, approaching those of firm power output at Hoover Dam, economy in sluicing head is a major consideration in design.

The multiple-basin plan seems to offer a more dependable and conservative design, from the standpoint of sluicing head, than that offered by the author. While no definite quantity has yet been determined as to the minimum required under the full capacity of diversion with the six basins in operation, the writer believes that 3 000 sec-ft would be quite adequate, if not much in excess of requirements under the heaviest silt content, in so far as cleaning of the basins is concerned. No doubt, larger heads will be necessary periodically to clear the river channel below the structure. Referring to Fig. 5 of the tentative plan shown by the author, using the width of 700 ft in the uniform approach channel and 8 ft as the depth of water in that part of the water cross-section above the horizontal division wall (or what might be called a "water splitter"), the resulting mean velocity of approach for 15 000 sec-ft of water becomes 2.68 ft per sec. Assuming 4 ft as a minimum depth of water below this wall, the water cross-section below the wall line produced and immediately up stream from the inlet to the under-sluices, is 2 800 sq ft. The mean velocity of approach in the full cross-section would be approximately at the location of the horizontal division (or about 0.7 ft above). The mean velocity in the lower part of the water section would depend somewhat upon the friction factor on the bottom of the approach channel, 1.7 ft being a safe minimum value to use. The mean velocity in the full water section would be approximately 2.4 ft per sec. Assuming that these velocities remain unchanged at the entrance to the under-sluices the discharge through the sluice-gates would be 4 760 sec-ft (neglecting the cross-section area of the longitudinal supporting walls and horizontal partition).

Considering inlet-gate capacity to the canal flume alone, the width of 700 ft, or 750 ft, as shown on the tentative plan, does not utilize all the 2.0 ft of head available. Using 0.6 ft as the approximate loss of head through trash

racks, the net velocity head would be 1.4 ft, resulting in an intake velocity of about 5.75 ft per sec. Using $7.5 \times 20 = 150$ sq ft, as gate-opening, the capacity of one gate is 860 sec-ft, and the total width of the gate structure would be more nearly 400 to 450 ft. The increased velocities resulting in a narrowed approach channel would be reflected in the lower section of the water cross-section. To compare the sluicing head, consider a 400-ft width of channel and of same water depth and like dimensions, of 8 ft above the partition line produced up stream and 4 ft below this line, the respective areas in section would be 3 200 and 1 600 sq ft. The mean velocities in the upper and lower sections would be 4.68 ft and in excess of 3.0 ft per sec, respectively. The resulting mean velocity in the full water cross-section would be about 4.2 ft per sec. Assuming, again, that these velocities were unchanged at the entrance to the under-sluices, the quantity of water passing under the partition and through the sluice-gates would be, conservatively, between 4 800 and 5 000 sec-ft. This comparison is not a suggestion that the width of the gate structure and approach channel in such a design should be limited to the minimum required for the maximum diversion quantity. Provision for fluctuation in pond depth above the structure would be necessary to provide against overtopping the overflow weir at low discharges, which is accomplished by a widened structure. The writer believes that the multiple-basin plan would prove to be more economical in sluicing water by an amount of between 2 000 and 3 000 cu ft per sec, under normal or low discharges.

There will be long periods when the diversion and under-sluices would be passing the full river discharge, and during such periods, if likewise long periods of unvaried inflow to the canal were maintained, such a design as the author suggests might function satisfactorily in segregating the bed load of soil. There will necessarily be fluctuations of heads to varying demands from irrigation and power, and these fluctuations must be controlled by co-ordination of operation of the intake gates to the flume and the under-sluice gates at the ends of the sluice culverts. Under most favorable conditions, in order to maintain undisturbed velocities at the intakes it would seem that each battery of gates for the full width of the structure would have to be operated as a unit. Any material variance in discharge through adjacent sluice-ways would result in a disturbance in approach velocities creating turbulence sufficient to put large quantities of the heavier materials into suspension and carried above the partition and into the flume. As a matter of practice it would seem that such a co-ordination of gate operation, with about fifty gates involved, would be most questionable of attainment.

Considering the effect of longitudinal supporting walls and the installation of trash racks at the entrance to the sluice-ways, each of such walls becomes virtually a "water splitter" in a vertical plane. If the horizontal wall could be permitted a considerable over-hang up stream, these might have but little effect upon the approaching water velocities. No over-hang of the horizontal wall would be permitted up stream from or above the trash racks. Therefore, any over-hang must necessarily support the racks from lateral pressure. The cantilever action in the over-hang would limit its length to

that of a short lip unless heavy beam reinforcement was added, which beam construction would be objectionable as an entrance factor. Submerged débris lodging against the trash racks above the under-sluices would, at best, be difficult to remove; but if lodged under any permitted over-hang it would be almost impossible to remove under flow conditions. It is thought that the combined effect of trash racks and vertical partition walls would be adverse to a continuous and unobstructed transportation to a free discharge of the bed soil.

Another serious consideration in the design of a structure of such proportions, particularly with a large number of gates, is the matter of maintenance. Accessibility to gates and sluice-ways in cases of repairs is necessary, and this with the minimum interruption of diversion and sluicing operations. The maintenance of the basin structure would be comparatively simple where it is possible to cut out one or more basins or sluice-ways without seriously impairing or interrupting the operation of the others.

The aggregate capacity of the tentative design for the dam and head-works by the Bureau of Reclamation is 259 000 sec-ft. It is unlikely that under regulated river flow the head-works will ever be called upon to utilize any quantity closely approaching that capacity. However, it is not unlikely that the river discharge at Imperial Dam will exceed, perhaps materially, 110 000 sec-ft prior to provision for other storage works below the Big Williams water-shed. From experience by the Imperial Irrigation District, the Yuma Project, and other agencies in dealing with the control of the Colorado River under flood discharges, it is hazardous to predict what may be the extent of shoaling and channelizing above any structure that tends to divide the flow of the river. It seems reasonable to believe that, under a sudden increase in the flow of the river from a comparatively low discharge to about 100 000 sec-ft, with a quickly receding river thereafter, that at times sufficient material would be deposited at the head of a trained approach channel to the diversion works to require removal before normal flow conditions could be restored, or at least to interrupt inflow to the diversion gates. In looking forward to the operation and maintenance of the head-works, naturally, the engineers of the Imperial Irrigation District are hopeful that the design eventually adopted will preclude any possibility of dredging operations.

At the time the location surveys were being made by the Bureau of Reclamation for the All-American Canal, the engineers of the Imperial District did some research work on certain of the main canals in determining hydraulic elements of flow. Of particular import was the adoption, as nearly as could be determined, of the proper value of the coefficient of n to be used in the design of canal sections. The canals studied were the Alamo Canal (the main canal through Mexico), at Alamo Mocho Station; the East Highline Canal at "B" Heading; the Central Main, Briar, and West Side Main Canals at the International Boundary Line; and the West Side Main Canal at what is known as the Drain Station.

In Table 1, the values of A, V, and Q were obtained from current-meter records of discharge and the values of p and r were computed. The slopes of the

TABLE 1.—STUDIES TO DETERMINE HYDRAULIC ELEMENTS OF FLOW

Date	Area, A, in square feet	Wetted perimeter, p, in feet	Hydraulic radius, r	Velocity V, in feet per second	Flow, Q, in cubic feet per second	Chezy coefficient, C	Kutter's n
(*a*) ALAMO CANAL, AT ALAMO MOCHO METER STATION, MEXICO; $S = 0.000255$							
January 6, 1926	621.4	121.5	5.11	3.25	2 025.6	89	0.0224
January 10, 1926	680.0	121.8	5.56	3.38	2 300.0	90	0.0225
February 11, 1926	661.5	122.9	5.37	3.78	2 498.1	102	0.0195
February 1, 1926	738.8	123.1	6.00	3.53	2 605.0	90	0.0230
December 6, 1926	632.8	125.3	5.05	4.58	2 899.8	127	0.0155*
October 12, 1926	619.0	126.9	4.86	5.00	3 100.8	142	0.0135*
March 4, 1926	675.5	123.4	5.48	4.73	3 199.8	127	0.0157*
June 7, 1926	695.5	124.1	5.60	4.74	3 296.5	125	0.0160*
May 23, 1926	844.5	123.9	6.80	4.14	3 499.9	100	0.0206
August 19, 1926	729.8	124.1	5.88	4.93	3 600.4	127	0.0158*
July 15, 1926	877.5	124.8	7.01	4.21	3 699.0	101	0.0210
July 8, 1926	872.0	123.4	7.06	4.55	3 965.4	107	0.0203
(*b*) EAST HIGHLINE CANAL, AT "B" HEADING; $S = 0.00322$							
January 7, 1928	187.0	54.7	3.42	2.16	405.6	65	0.028
December 25, 1928	182.5	54.5	3.35	3.25	593.5	99	0.0186
November 19, 1928	203.6	55.4	3.68	3.47	705.1	101	0.0183*
January 28, 1928	221.1	56.0	3.94	3.62	801.2	102	0.0183*
October 10, 1928	234.7	56.5	4.17	3.83	900.4	104	0.0183*
October 27, 1928	256.2	57.5	4.45	3.90	1 000.9	103	0.0185*
October 21, 1928	269.8	58.3	4.61	4.10	1 100.6	106	0.0183*
January 28, 1928	270.6	58.0	4.66	4.45	1 201.6	113	0.0175
September 4, 1928	300.7	59.6	5.03	4.35	1 301.6	108	0.0183*
May 23, 1928	298.4	59.7	5.00	4.69	1 399.7	117	0.0172
July 28, 1928	348.7	62.1	5.60	4.58	1 598.5	108	0.0183
(*c*) CENTRAL MAIN CANAL, BOUNDARY METER STATION, CALIFORNIA; $S = 0.00044$; MAXIMUM CAPACITY, 1 050 SECOND-FEET							
April 13, 1928	52.40	38.08	1.38	1.94	101.85	79	0.0205
January 8, 1928	82.80	40.87	2.03	2.47	204.11	83	0.0205
March 25, 1928	120.80	43.69	2.77	3.31	399.24	95	0.0188
January 26, 1928	133.40	43.96	3.03	3.02	402.35	83	0.0218
July 1, 1928	145.70	44.83	3.25	3.45	502.87	91	0.0200*
September 21, 1928	157.70	45.49	3.47	3.81	600.14	97	0.0195*
May 4, 1928	181.80	46.80	3.88	3.87	703.83	94	0.0200*
August 4, 1928	189.40	46.79	4.05	3.96	750.69	95	0.0200*
June 8, 1928	218.2	48.71	4.48	4.15	905.84	93	0.0200
(*d*) WEST SIDE MAIN CANAL, BOUNDARY METER STATION, CALIFORNIA; $S = 0.000525$							
1928	59.2	27.2	2.18	2.02	119.61	59	0.033
1928	144.2	36.7	3.93	2.76	399.29	61	0.031
1928	190.8	40.5	4.71	3.13	600.1	63	0.032
1928	225.6	41.8	5.38	3.34	751.3	62	0.032
(*e*) WEST SIDE MAIN CANAL, DRAIN METER STATION, CALIFORNIA; $S = 0.00038$							
March 18, 1929	134.00	32.65	4.10	2.26	303.22	57	0.035
January 20, 1929	136.00	29.60	4.59	2.54	345.57	61	0.033
January 7, 1929	149.60	33.90	4.41	2.68	401.62	63	0.0315
February 15, 1929	149.60	34.05	4.39	2.68	400.35	66	0.030
August 28, 1929	156.50	34.10	4.59	2.89	451.00	71	0.0275*
August 5, 1929	164.60	35.30	4.66	2.90	476.91	69	0.027*
July 12, 1929	163.80	35.95	4.55	3.06	500.48	74	0.026*
August 22, 1929	166.90	34.95	4.78	3.07	512.66	72	0.0275*
May 6, 1929	170.20	35.35	4.81	3.11	530.00	73	0.027*
May 10, 1929	172.50	36.50	4.74	3.20	552.30	75	0.027*
May 11, 1929	182.60	36.65	4.98	3.33	601.56	76	0.027*
May 21, 1929	185.30	36.85	5.02	3.34	619.92	76	0.027*
June 21, 1929	190.40	37.15	5.12	3.42	651.03	77	0.026*
(*f*) BRIAR CANAL, AT INTERNATIONAL BOUNDARY LINE; $S = 0.000382$; MAXIMUM CAPACITY, 250 SECOND-FEET							
January 1, 1928	69.7	26.45	2.63	1.79	125.26	56	0.0300
October 30, 1928	73.6	27.75	2.66	2.10	155.0	73	0.0245
May 17, 1928	62.6	27.60	2.26	2.72	170.4	92	0.0186
April 22, 1928	59.6	27.60	2.16	3.02	180.1	105	0.0162*
June 28, 1928	72.0	27.75	2.60	2.66	191.3	84	0.0210
March 25, 1928	67.0	28.20	2.37	2.98	200.3	99	0.0177*
September 3, 1928	64.4	27.95	2.30	3.11	200.4	105	0.0165*
June 18, 1928	65.6	28.72	2.28	3.04	200.4	103	0.0168*
July 10, 1928	71.7	28.75	2.49	3.00	215.0	87	0.0198*
July 11, 1928	80.6	29.35	2.75	2.73	220.8	85	0.0207
August 7, 1928	79.0	29.50	2.68	2.90	229.2	90	0.0195*

*Sections considered non-silting and non-scouring.

canals were determined by field measurements and C was computed from the Chezy formula: $V = C\sqrt{rs}$. The values of n were determined from a diagram of functions computed from Kutter's formula. Considerable thought was given in these studies to the determination of non-silting and non-scouring velocities.

At Alamo Mocho Station, the Alamo Canal is known to silt and scour more or less. No doubt, this explains the erratic values in Table 1 (*a*), which shows a very low value of n at medium discharges and high values at high and low discharges, when silting and scouring, respectively, occur.

Referring to Table 1 (*b*), it is to be noted that the results show a very consistent and relative increase of velocity, with the hydraulic radius. They also show a quite consistent value of the friction coefficient, say, a mean of 0.0185. In Table 1 (*c*), the variable n-values for flows between 100 and 400 ft per sec, are probably due to the effect of silting at low velocities. The mean value of n in Table 1 is approximately 0.020.

The West Side Main Canal just below the meter station at the International Boundary should not be considered typical of a non-silting and non-scouring canal. The sides and bottom of this canal show a hard and tight clay, and an eroded rough surface. This results in the unusually high coefficient of friction as evidenced by Table 1 (*d*).

At the Drain Meter Station, the West Side Main Canal is practically self-maintaining. What silting may take place at low heads is relieved when these heads increase, and no cutting of the banks takes place at maximum heads. It is to be noted in Table 1 (*c*) that the functions are quite consistently related through a range of flow from 450 to 650 cu ft per sec. The variation of Kutter's n is also uniform.

The Briar Canal functions under a light grade and comparatively slow velocities, which tend to permit the rapid deposition of silt under normal heads. The Briar Sluice at Birch Canal was installed after the meterings listed in Table 1 (*f*) were taken. This canal should not be considered typical of those that are non-silting and non-scouring.

The action of bed silt is interesting in the fact that at times the meter gauges will indicate a smooth even cross-section and then, again, large holes are found across the bottom of the canal caused by a movement of the bed silt. It has also been noted that even where a canal has been considered self-cleansing there are times when large quantities of bed silt will move in and raise the bottom considerably. This will last sometimes for only a day or two and again for a period of several weeks. This movement of bed silt creates large fluctuations in canals of the size of the Alamo and makes it difficult to maintain a regulated flow.

From Table 1 it will be noted that these silt-laden canals of the Imperial System function under high velocities relative to earth canals. This is true in cases for discharges as low as 50% of the canal capacities. Close observations of the functioning of these canals under varying heads indicates that under flows exceeding about two-thirds of the full capacity, little trouble is experienced in the silting in or scouring out of the beds of the canals,

particularly those operating under excessive grades. Relatively high velocities are required, also, to maintain movement of the bed load, which is often very erratic.

Information that is helpful in the consideration of velocities in a channel of approach to a desilting structure, whether it be to function automatically, by settling basins, or by mechanical means, might be gained from a study of Table 1. The writer is of the opinion that it would require a mean velocity closely approaching 4 ft per sec for an unlined channel of 15 000 to 20 000 sec-ft capacity, in order to keep the bed load of silt moving with a sufficient degree of uniformity. Mean velocities in excess of $4\frac{1}{2}$ ft per sec would tend to turbulency. These deductions are also based on the results made from similar studies of the velocities in the Colorado River made by the Bureau of Reclamation from gauging records of the Yuma Project at the time the canal studies were made. If correct, these deductions would be adverse to the acceptance of eliminating entirely the basins in favor of under-sluice operation where so wide a channel is involved.

At the present writing (1933) engineers of the Yuma Project are conducting certain silt studies at the project diversion at Laguna Dam. Most certainly, as the author suggests, the correct solution of the problem in the design of the Imperial Diversion warrants sufficient research work along this line.

R. F. WALTER,[40] M. AM. SOC. C. E. (by letter).[40a]—The author has submitted a valuable and opportune paper, which is of particular interest to the writer due to his connection with the U. S. Bureau of Reclamation, to which organization has been entrusted the duties of designing and constructing the works discussed. The structure referred to, and for which tentative plans are shown in Figs. 5, 6, 7, and 8, has been officially designated, Imperial Dam. With its appurtenant works, it will constitute the diversion and desilting structure for the All-American Canal. The design and construction of these works now (1933) await certain legal actions and the appropriation of the necessary funds by Congress.

In his acknowledgment, Mr. Rothery makes reference to certain material for the preparation of Figs. 5, 6, 7, and 8 of his paper, as having been made available through the writer. The material referred to is a drawing showing a desilting plan very similar to that shown by Fig. 8 of the author's paper, but with the sand sluice omitted. Reference is also made in the paper to a cost of $3 375 000 for the proposed desilting works. Both the cost estimate and the drawing referred to by the author, are part of a report by one of the engineers of the Bureau of Reclamation on the All-American Canal investigations carried on during 1929 and 1930. The report is dated May, 1931.

While it was doubtless not the author's intention to give the impression that the plan furnished him or the estimate of cost were other than tentative, still the impression might readily be gained that they represented works

[40] Chf. Engr., U. S. Bureau of Reclamation, Denver, Colo.

[40a] Received by the Secretary March 22, 1933.

designed and approved for construction. This is not the case since the plan which was the basis for the estimate is of a very preliminary and tentative nature.

The investigations leading to the report of May, 1931, were conducted in the same manner as other similar investigations of the Bureau of Reclamation. Only sufficient field surveys and office designs were made to permit of reasonably dependable estimates of cost being prepared. For such a preliminary report and estimate it was obviously not practicable to go carefully into the design of a structure of such uncertain operating characteristics. Instead, and as a basis for a tentative estimate of cost, a sketch plan was made for a structure somewhat similar in its operation features to the Laguna Dam at the head of the Yuma Canal. The desilting plan at this dam, while being far from ideal or of great efficiency, has served the Yuma Project for many years.

Reference is made by Mr. Rothery, to the methods used by the Imperial Irrigation District to prevent, in so far as possible, the bed load of the Colorado River being carried through the canal. This has proved to be an expensive undertaking and it is known that the District's officials consider even the Laguna Dam method of silt removal a great improvement over the method used by the District, and by comparison quite satisfactory and economical in operation.

Probably one of the most objectionable features of the desilting basin at Laguna Dam is the necessity of having to interrupt irrigation deliveries during the sluicing period. This is necessary because there is only one basin and no by-pass. To obviate this necessity, and in order to carry the greatly increased capacity of the All-American Canal, six desilting basins were tentatively proposed for construction at Imperial Dam. With this system the full capacity of the canal could be passed slowly through any five basins while the sixth was being sluiced of its accumulation of sludge without interrupting full canal diversion. For less than full canal capacity (which will doubtless be the usual operating condition for many years after the canal is put into operation, and even then for the greater period of each year) less basins could be utilized, but probably a better plan still would be to utilize five basins, in which case the flow through the basins would be at a reduced rate, permitting better settlement conditions. For velocities of less than 0.5 ft per sec appreciable quantities of materials that are on the border line between bed and suspended load should settle out in the basins.

Whether quantities of suspended material, such as would justify the large basins tentatively considered, will settle out with the depths necessary in the basins and at velocities of flow required, should be known definitely when the very carefully planned investigations now being carried on at Laguna Dam are completed. These investigations should demonstrate quite definitely the efficiency and economic practicability of this type of basin. Due to the availability of suitable materials—earth for the embankments and rock for rip-rap—the excess cost of this type of desilting works will not be as great as might at first be assumed, because of the long embankments required in the construction of the basins, compared with the cost of the more simple sluicing structure, shown in Figs. 5 and 6. However, should the present

investigations show that no great efficiency is gained by building the large basins they should not be used.

It is not the writer's intention to enter into a discussion of the comparative merits of the desilting plan upon which the estimate of the Bureau of Reclamation was based, or of the simplified desilting structure suggested by the author, since certain important investigations being conducted by the Bureau which are expected to furnish the fundamental data for the preparation of final designs, have not yet been carried to such a point as would justify such a discussion. However, several different plans for the exclusion of silt from the canal, both bed and suspended load, have been, and are, being considered by the Bureau of Reclamation.

The general plans considered cover a wide range of construction, from the most simple sand-trap structure of unknown efficiency, to a comprehensive system of mechanical clarifiers of relatively known efficiency, such as are used in connection with municipal water purification plants and certain mining operations. The so-called vortex tube, with its various modifications, has also been given consideration for the removal of bed silt that it may not be possible to exclude from the head of the canal or that, for one reason or another, may reach the canal below its point of diversion.

The difference in first cost of the various plans considered is naturally very great and not directly comparable since the performance expected from each plan is quite different. One familiar with the silt problem of the Colorado River will realize at once that the cost of desilting works will increase rapidly with any requirement for increased efficiency in the removal of the suspended silt transported. The final choice of plan may not be based purely on first cost, or on operating cost, since the question of efficiency in operation and desirability as regards the quantity of suspended silt to be removed, may have a deciding influence in the choice.

Each succeeding year brings out more clearly the detrimental effects, to the present irrigated areas of the Yuma Project and the Imperial Irrigation District, of irrigation with water containing large quantities of suspended and colloidal materials. Water users have been known to have closed their headgates and to do without water for limited periods, rather than to accept water especially heavily laden with fine silt. Whether it is economically possible to remove such materials or whether the construction of Hoover Dam, and possibly Parker Dam, will automatically correct this undesirable condition within a reasonable period of years, are matters for consideration.

The author's suggestion that the silt carried in suspension, during the early years of canal operation, will be of benefit to new land and lateral developments, and should be taken advantage of in connection with the development of new lands with sandy soil, may have some merit; unfortunately, however, it is not likely that any great area of the new sandy lands will have been brought under cultivation until after the stabilizing effects in the river shall have become felt and the quantity of silt carried in suspension will have been greatly reduced.

The writer is not in full agreement with the author on all the points discussed in his paper, but on one point at least he is in full agreement; that is,

wherein he suggests that due to the permanence and importance of the control structure, competent investigations and research pertaining to the silt problem should be made so that as much as possible of the present guesswork will be eliminated.

In line with this suggestion the Bureau of Reclamation has already made an exhaustive research of published literature in this as well as in many foreign countries, in an effort to have available all data bearing on the silt problem. Access has also been had to many records and reports not available in published form. A bibliography of all such information has been prepared and is being added to as rapidly as new data become available.

Records have been kept of the quantities of silt carried in suspension in the Colorado River at Yuma, Ariz., for a period of more than eighteen years. Records of silt content at other points on the river are also available. These data as regards the content of suspended silt are considered quite adequate and dependable for making a comprehensive silt study. However, as regards bed load little definite information is available. Unfortunately, to date, no positive means are known to have been developed for measuring the bed load carried by streams. Study is being given this matter and plans are now (1933) under way which it is hoped will give reasonably accurate measurements of the quantity of total silt load in the Colorado River in the vicinity of the All-American Canal diversion. While positive determination of the bed load may not be possible, it is confidently believed that results can be obtained that can be used in the design of desilting works with greater assurance of reliability.

As stated previously, carefully planned investigations are being made at Laguna Dam in an attempt to learn additional facts regarding silt in the Colorado River. Analysis of the silt carried in suspension, as well as that deposited in the desilting basin, is being made. By use of the hydrometer method, determination is being made, not only of the particles susceptible of analysis by the ordinary sieve method, but of sizes down to those that require as much as two hours for settlement in still water in short tubes. It is hoped that determination can be made of the rates of settlement of the various sizes of silt particles, both in still water and at rates of flow such as may obtain in the All-American Canal, or in whatever type of desilting basin is under consideration.

A somewhat different and more comprehensive program of investigation and laboratory experimentation has already been outlined, but this can be undertaken only in case a special appropriation for the work is made available. During these investigations it is hoped to be able to ascertain with some degree of accuracy the quantity of silt that will be carried to the Imperial Dam, under flow conditions such as will obtain in the river after the construction of the Hoover and Parker Dams, and with the regulated flows required for diversion and sluicing the river channel and keeping it cleared of the sludge that will be turned back to the river from the desilting structure. Under regulated flow conditions in the river, discharges greater than 25 000 sec-ft will probably be of rare occurrence, within a few years after the

completion of Hoover Dam. Discharges of less than this quantity, even with clear water released from the reservoirs, may not carry the quantity of bed load anticipated.

The author has called attention to an important problem in the development of the All-American Canal System, and one that must be given a great amount of consideration. Another problem, or rather a part of the same problem, that has been given little consideration in his paper, is that of the disposal of the sludge returned to the river from the partly desilted water diverted to the canal. This may prove to be one of the most discouraging problems of all. However, in some respects, the conditions will probably be favored by the natural occurrences in the sequence of operation of the Hoover Dam and the All-American Canal. For example, during the early stages of operation of the canal, only partial capacity will be required for the purpose of irrigation and power development along the canal. This will call for desiltation of a reduced quantity of water with a resulting decrease in the quantity of sludge returned to the river. Furthermore, a relatively large river discharge may be available, during the first few years of canal operation, for use in sluicing the sludge discharge down the river. Another factor that will also tend to diminish, possibly quite materially, the sludge return to the river below the Imperial Dam, will be the construction in the river of this dam, the crest elevation of which will probably be nearly 25 ft above the present elevation of the river bed. The dam will create a temporary reservoir of appreciable area that will act as an effective settling basin during the early period of canal operation. On the other hand, it may be that during the first few years of river re-adjustment, after completion of the Hoover and Parker Dams, relatively high river discharges will still occur and even greater quantities of bed load materials than at present will be carried to the Imperial Dam.

HARRY F. BLANEY,[41] ASSOC. M. AM. SOC. C. E. (by letter).[41a]—The author has contributed a valuable paper on the subject of silt transportation. It is of special interest at this time because large sums of money will be spent during the next few years building structures to utilize the waters of the Colorado River, and the success of such projects will depend primarily upon whether diversion works are properly designed for silt control. The paper brings out clearly the fact, not always recognized in the past, that the bed load in the river is large and that the control of bed silt is one of the major problems confronting the designer of head-works on the Lower Colorado.

In 1926, Samuel Fortier, M. Am. Soc. C. E., and the writer prepared a report[42] on a series of silt investigations on the Colorado River, conducted by the U. S. Department of Agriculture from 1907 to 1925. One of the surprising facts revealed in these studies was that the suspended silt load in the river at Topock, Ariz., was considerably greater than that at Yuma, Ariz., 206 miles farther down stream. Comparison of the normal annual load at Topock

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[41] Irrig. Engr., Bureau of Agri. Eng., U. S. Dept. of Agriculture, Los Angeles, Calif.

[41a] Received by the Secretary March 29, 1933.

[42] "Silt in the Colorado River and Its Relation to Irrigation," by Samuel Fortier and Harry F. Blaney, *Technical Bulletin No. 67*, U. S. Dept. of Agriculture, 1928, p. 53.

with that at Yuma, exclusive of the Gila River, indicated that 36% of the silt at Topock either passed Yuma as bed silt or was temporarily deposited in the channel or flood plains above Yuma.[42] An investigation by Mr. C. S. Howard[43] confirms this finding. The average of his results for a 3-year period ending September 30, 1928, shows that the annual suspended matter carried by the river was 286 000 000 tons at Grand Canyon, 231 000 000 tons at Topock, and 174 000 000 tons at Yuma. After applying a correction factor by estimating the inflow of the Gila River above the Yuma Station, these data indicate that about 34% of the suspended silt load at Topock became bed silt before reaching Yuma. The difference between the average annual quantity of suspended silt measured at the Grand Canyon and that at Yuma, exclusive of the Gila, is estimated as 72 000 acre-ft, based on the assumption that 12% of the silt load at Yuma comes from the Gila River,[42] and that the dry weight of silt is 85 lb per cu ft. Undoubtedly, most of this becomes bed silt.

Thus far, no method has been developed by which the volume of material moved as bed load can be measured. The aforementioned calculations may be a means of approximating the quantity of bed load carried by the river past the diversion works contemplated for the All-American Canal. The writer is of the opinion that as the river emerges from the canyon section, it is carrying its greatest load of suspended silt due to the steeper grades, higher velocity, and churning effects of the canyon section; then as the river flows on flatter grades below the canyon section the heavier silt is temporarily deposited and becomes bed silt, or rests on the bed of the channel until a flood carries it farther down stream. Many estimates have been made of the volume of silt transported by the Colorado River and some of them are shown in Table 2.

TABLE 2.—SOME ESTIMATES OF SILT TRANSPORTED ANNUALLY BY THE COLORADO RIVER

References	Location	Period	Dry weight of silt per cubic foot, in pounds	Annual Silt Load, in Acre-Feet		
				Suspended	Bed	Total
Dole and LaRue*	Yuma, Ariz.	1895–1914	93	80 000	......	....
Mead, Schlecht, and Grunsky†	Yuma, Ariz.	Average	100	90 000	12 000	102 000
Weymouth‡	Yuma, Ariz.	1909–1922	86	105 000	......	....
Davis§	Yuma, Ariz.	Average	85	113 000	......	....
Davis§	Boulder Canyon.	Average	...	88 000	......	....
Rothery‖	Yuma, Ariz.	1912–1921	86	94 800	......	....
Fortier and Blaney¶	Yuma, Ariz.	Average	...	119 000	19 000	138 000
Fortier and Blaney¶	Boulder Canyon.	Average	85	....	......	137 000
Grunsky**	Yuma, Ariz.	1911–1927	100	94 500	......	....
Grunsky**	Yuma, Ariz.	Average	100	....	25 000 to 30 000	....
Howard††	Grand Canyon.	1925–1928	86	152 700	......	....

* *Water Supply Paper 395*, U. S. Geological Survey, 1916, p. 222. † Report of All-American Canal Board, 1920, p. 25. ‡ "Colorado River Development", Senate Doc. 186, 1929, p. 52. § "Problems of Imperial Valley and Vicinity", Senate Doc. 142, 1922, pp. 3-4. ‖ *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1419. ¶ "Silt in the Colorado River and Its Relation to Irrigation", *Technical Bulletin No. 67*, U. S. Dept. of Agriculture, 1928, pp. 61–62. ** *Transactions*, Am. Soc. C. E., Vol. 94 (1930), pp. 1118, 1126. †† "Suspended Matter in the Colorado River in 1925–1928", *Water Supply Paper 636–B*, 1929, p. 28.

[43] "Suspended Matter in the Colorado River in 1925-1928," by C. S. Howard, *Water Supply Paper 636-B*, U. S. Geological Survey, p. 24.

The author gives the reader the impression that "the traveling suspended load cannot be greatly reduced with the desilting basins." It is true that desilting has not been so successful at the Rockwood Heading of the Imperial Irrigation District, due primarily to the fact that the river is not regulated and practically the entire flow is diverted into the heading at extremely low stages of the river. However, experiments made in 1918 by the Bureau of Agricultural Engineering on the efficiency of desilting at the head-works of Laguna Dam, indicated that the desiltation ranged from 33 to 72% and averaged about 57 per cent.[44] Raymond A. Hill, M. Am. Soc. C. E., also carried on experiments at intervals during several years, and he found that desiltation ranged from 18 to 70% with an average of 50 per cent.[44]

While the intake at Laguna Dam for the Federal Project at Yuma, is the most effective that has been installed to date on the Colorado River, in ridding the water of its heavier silt it is creating a soil problem that may be difficult to solve. By the desilting process the clays and colloidal material are separated from the sand in the silt and deposited on the irrigated land. This annual deposition is bringing about a change in the character of the surface soil, making it more sticky, more difficult to cultivate, and more impervious.

The writer believes that the term, "silt," is preferable to the term, "soil," in the title of the paper, since the term, "silt," is now more commonly used by engineers and is subject to less criticism by soil authorities than in the past.

CALVIN V. DAVIS,[45] M. AM. SOC. C. E., AND DIRK A. DEDEL,[46] ASSOC. M. AM. SOC. C. E. (by letter).[46a]—This paper describes excellent designs for structures to remove silt and bed load, which are appurtenant to the proposed Colorado River diversion dams. In view of the thoroughness with which the author has treated all phases of this problem, it is difficult to suggest basic improvements in his tentative plans. Several individual elements of the plans, however, may be discussed to advantage, owing to the fact that alternate designs capable of performing the same functions may be used.

In Fig. 6 (*a*) and Fig. 8 (*a*), the author has shown, up stream of the diversion works, intake tubes that consist of a floor and a horizontal partition supported by light vertical walls. These intake tubes are intended to convey a major portion of the traveling bed load to conduits which either pass directly under the dam and flume and discharge in the river down stream of the flume, as shown by Fig. 6 (*a*), or turn at right angles at the toe of the intake structure and discharge in the river below the main diversion dam, as shown by Fig. 8 (*a*).

The writers are of the opinion that while the intake tubes, as shown in Fig. 6 (*a*) and Fig. 8 (*a*), will trap most of the moving bed load there is the

[44] "Silt in the Colorado River and Its Relation to Irrigation," *Technical Bulletin No. 67*, U. S. Dept. of Agriculture, p. 58.

[45] Chf. Designer, Ambursen Dam Co., New York, N. Y.

[46] Designing Engr., Ambursen Dam Co., New York, N. Y.

[46a] Received by the Secretary April 5, 1933.

possibility that under certain flood conditions a part of this bed load may pass into the canal.

A modification of Mr. Rothery's intake design which the writers believe will, under all flood conditions, eliminate the traveling bed load and most of the suspended silt load before it reaches the canal, is shown by Fig. 9,

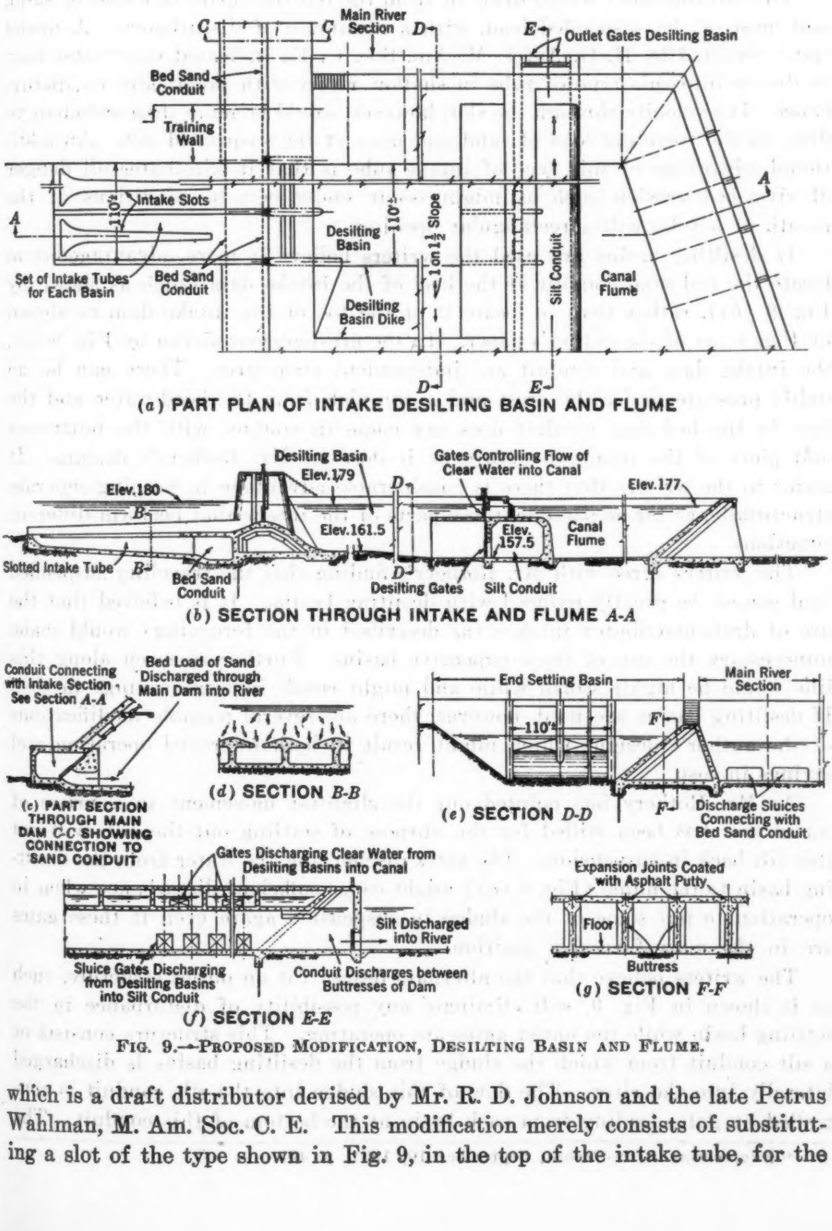

FIG. 9.—PROPOSED MODIFICATION, DESILTING BASIN AND FLUME.

which is a draft distributor devised by Mr. R. D. Johnson and the late Petrus Wahlman, M. Am. Soc. C. E. This modification merely consists of substituting a slot of the type shown in Fig. 9, in the top of the intake tube, for the

horizontal partition and rectangular entrance as indicated by Mr. Rothery. The tube shown by Fig. 9 (*b*), is connected to a bed-sand conduit at the heel of the intake dam. This conduit is carried across to the heel of the dam in the main river section where its discharge into the river is regulated by sluice-gates (Fig. 9 (*c*)).

This arrangement would draw in from the top the entire bed load of sand, and most of the suspended load, with a minimum of disturbance. A model test,[47] reported by H. G. Acres, M. Am. Soc. C. E., indicated that water may be drawn into this type of tube in shallow rivers with practically no disturbance. The velocity through the slot, however, would be more than sufficient to draw in the traveling load of sand and most of the suspended silt. An additional advantage of this type of intake tube is that it eliminates all danger of river-bed erosion such as might occur under certain conditions at the mouth of a tube wth a rectangular opening.

If desilting basins are used the writers believe it more advantageous to locate the bed-sand conduit at the heel of the intake dam (such as shown by Fig. 9 (*b*)), rather than to locate it at the toe of the intake dam as shown by Fig. 8 (*a*) of the author's paper. In the arrangement shown by Fig. 9 (*b*), the intake dam and conduit are independent structures. There can be no uplift pressure under the crest and apron slab from the head-water and the flow in the bed-sand conduit does not come in contact with the buttresses and piers of the intake structure as it does in Mr. Rothery's designs. It seems to the writers that there is considerable advantage in keeping separate structurally, as far as possible, those parts of the intake that perform different functions.

The writers agree with Mr. Rothery's finding that the traveling suspended load cannot be greatly reduced with desilting basins. It is believed that the use of draft-distributor intakes (as described in the foregoing) would make unnecessary the use of these expensive basins. Further research along this line would be highly worth while and might result in large savings in cost. If desilting basins are used, however, there are several possible modifications of the author's designs which might result in both improved operation and savings in cost.

As Mr. Rothery has pointed out the slightest movement in a prism of water that has been stilled for the purpose of settling out the silt, will put this silt back in suspension. The gates discharging the water from the desilting basin to the flume (Fig. 8 (*a*)) might cause sufficient disturbance when in operation to put some of the sludge in suspension again even if these gates are in the most favorable position.

The writers believe that the alternate design for an outlet structure, such as is shown in Fig. 9, will eliminate any possibility of disturbance in the settling basin while the outlet gates are operating. This structure consists of a silt conduit from which the sludge from the desilting basins is discharged laterally into the river. The flow of this sludge into the silt conduit is controlled by gates leading from each basin at the bottom of this conduit. The

[47] *Engineering News-Record*, September 10, 1931, p. 403.

flow of clear water from the desilting basin to the canal is controlled by a small gate structure on top of the conduit. In front of these gates are horizontal partition slabs supported by light vertical walls which are somewhat similar to those used by Mr. Rothery in front of the intake tubes. These horizontal slabs are intended to prevent the higher velocity adjacent to the gate from causing any disturbance in the desilting basin after the sludge has settled.

The flow through the silt conduit is controlled by outlet gates in the outer partition wall of a dam of the buttress type supported by a floor-slab. Two of the buttresses of this dam are continuous with the walls of the silt conduit, as shown by Fig. 9 (*a*) and Fig. 9 (*f*). This type of buttress dam has been shown for both the end of the desilting basin and the sides of the canal flume.

It is believed that the arrangement of the silt conduit shown by Fig. 9 will be more enonomical than the proposal to discharge the sludge from the desilting basin through conduits under the flume, as shown by the author. The expensive conduits are omitted, and only light paving is required for the bottom of the flume.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

MODEL LAW FOR MOTION OF SALT WATER THROUGH FRESH

Discussion

By John B. Drisko, Jun. Am. Soc. C. E.

John B. Drisko,[11] Jun. Am. Soc. C. E. (by letter).[11a]—This paper is another striking example of the versatility of hydraulic models. It is also of great value in showing the necessity of a proper choice of model scales if correct results are to be obtained.

The authors introduce their Equation (1) as being equal to "the change of momentum, MV_o, per sec", thus ascribing the velocity of the wave front, $V_o = \frac{dx}{dt}$, to the entire moving mass. This is a bold assumption. If it had been assumed that the velocity decreased slightly from the wave front back toward the undisturbed liquid (that is, in Fig. 6, from B toward Plane C-C), then the momentum of the moving slug $(= \int v\, dM)$ would have been less, yielding a higher value of V_o in Equation (5). If a higher value of V_o is correct, then the apparent agreement between the observed results and the (low) values of Equation (5) may be due to the fact that the observed "initial velocity" was measured over a distance long enough to permit of considerable viscous retardation.

Furthermore, if the authors' assumption regarding the velocity is retained, then the kinetic energy of the entire moving mass is,

$$\tfrac{1}{2} M V^2_o = \tfrac{1}{2} x\, d\, \frac{w}{g} (G_1 + G_2)\, V^2_o$$

Equating this to the decrease in potential energy (which is $< \frac{1}{4}$ $w\, s\, d^2_x$ since the center of gravity of the salt-water slug, $C\ O\ B\ C$, has dropped from a height, $\frac{d}{2}$, to a height greater than $\frac{d}{4}$) gives a result that is quite

Note.—This paper by Morrough P. O'Brien and John Cherno, Assoc. Members, Am. Soc. C. E., was published in December, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1933, by W. E. Howland, Assoc. M. Am. Soc. C. E.; and April, 1933, by Messrs. Herbert D. Vogel, and C. E. Grunsky.

[11] Instr., Civ. Eng., Mass. Inst. Tech., Cambridge, Mass.

[11a] Received by the Secretary February 20, 1933.

independent of x, namely, $V_0 < \sqrt{\frac{1}{2}\,\frac{g\,d\,s}{(G_1 + G_2)}}$. This does not agree with observation or with previous calculation and, therefore, the average velocity of the entire moving slug must be less than V_0.

It is interesting to note that Equation (5) is merely the usual equation for the velocity of a wave of translation, $v = \sqrt{gh}$, with the term, $\frac{s}{(G_1 + G_2)}$, included to compensate for the conditions of specific gravity and for the fact that both salt water and fresh water are being accelerated. The wave height, h, is, in this case, equal to $\frac{d}{2}$. Whether or not the absolute value of V_0 calculated by the authors is correct, the form of the expression seems rational, and hence the validity of the model law as derived is in no way affected.

The authors describe the action of a lock full of salt water when released into a larger body of fresh water. The unbalanced force, before release, is the excess of the static pressure on the gate. The distribution of this excess pressure is as shown in Fig. 15. It would be interesting to investigate the reverse condition in which the unbalanced force is in favor of the fresh water, as shown in Fig. 16. As before, the pressure distribution is triangular, but the base of the pressure triangle is at the surface of the salt water instead of at the bottom. The extra height of the fresh water shown could be

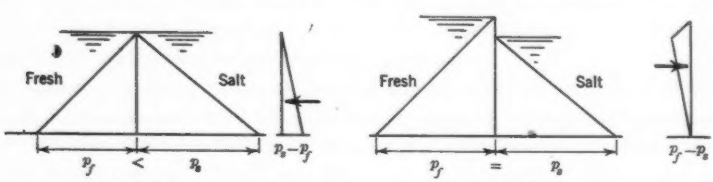

FIG. 15 FIG. 16

replaced by a restrained float, in order to reduce to a minimum the quantity of fresh water that would immediately flow over the surface of the salt water.

The statement of the authors, in the last paragraph under "Theory" that "* * * a drop of the surface at any place would have to be made up by a rise of the surface at some other place; * * * and, therefore, no drop of the surface could furnish the energy required to set water in motion", is confusing. If salt water drops and fresh water rises, there is a net loss of potential energy which is available for conversion to kinetic energy.

The derivation of the model law is clear and logical. Of particular significance is the statement following Equation (13) that its physical meaning is that "the inertia, pressure, and friction forces in the prototype must be reduced in the same ratio in the model." Inertia, pressure, and friction are the major forces in this particular problem, and, for true similarity, they must all be given due regard. With some types of models it is difficult or impossible to represent two types of forces in the same model. In the case of ship models, for instance, the skin friction resistance and the wave or

form resistance vary according to different laws. In testing ship models, the total resistance is measured, the frictional resistance is calculated empirically, and the difference gives the form resistance of the model. If two types of forces cannot be properly represented in the same model, two models constructed to different scales are sometimes used. In each model, one particular phase of the problem is studied, with little or no regard to the other.

In this paper, the authors have shown how all the essential conditions for a proper simulation of the motion of salt water through fresh may be represented in a single model which will yield correct results.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

WIND STRESS ANALYSIS SIMPLIFIED

Discussion

BY MESSRS. A. FLORIS, ROBINS FLEMING, J. D. GEDO, H. V. SPURR, AND JOHANNES SKYTTE

A. FLORIS,[22] ESQ. (by letter).[22a]—In this interesting paper the author analyzes the wind stresses in the frames of tall buildings by the aid of the Cross method of balancing fixed-end moments. Furthermore, he states that an exact analysis of these frames requiring the solution of simultaneous equations — as, for instance, the slope deflection method used by Wilson and Maney — is a rather tedious process. For this reason he proposes the application of a less accurate method of moment distribution to the analysis of multiple-story frames subjected to wind pressure.

The present analysis, based as it is on the repeated distribution of the end moments, is to some extent a trial-and-error method. Such indirect methods, however, require considerable time, and their application to practical problems is rather discouraging. Their use can only be justified if direct and speedy methods of analysis are not available.

In the present case this is not necessary, because a direct method[23] has been developed by Professor F. Takabeya which excels in brevity and accuracy all other methods known to the writer. The necessary elastic equations derived by the slope deflection method can be written almost automatically and solved by the principle of iteration. This method of solving simultaneous equations permits the use of the slide-rule and is convenient and simple.

The author's statement regarding the difficulties arising in the analysis of multiple-story frames subjected to an additional translation of the joints, is not quite correct. Such frames are analyzed quite easily by the use of the

NOTE.—The paper by L. E. Grinter, Assoc. M. Am. Soc. C. E., was presented at the meeting of the Structural Division, New York, N. Y., January 19, 1933, and published in January, 1933, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1933, by Messrs. Francis P. Witmer, Elmer K. Timby, N. A. Richards, John B. Letherbury, Frederick Martin Weiss, and Raymond C. Reese.

[22] Civ. Engr., Los Angeles, Calif.

[22a] Received by the Secretary March 11, 1933.

[23] "Methode der Gleichungstabellen zur Berechnung der Rechteckrahman," von Fukehei Takabeya, *Proceedings*, World Eng. Congress, Tokyo, 1929, Vol. VII, Pt. I, p. 277; also, "Rahmentafeln," von Fukehei Takabeya, Berlin, 1930, and "Das Verfahren der Gleichungstabulierung zur Berechnung des durchlaufenden Rahmens mit verschieden hohen senkrechten Stielen," von F. Takabeya, *Der Bauingenieur*, 1933, pp. 92 and 126.

Takabeya method, for instance, with little additional labor, and the calculations are not difficult. Furthermore, the claim that the method of balancing fixed-end moments only involves simple arithmetic is not quite true. In the last analysis any method arranged for practical use requires the application of simple arithmetical processes. For this reason the writer believes that, as an important practical advantage of the method under consideration, this fact is over-emphasized by the author.

ROBINS FLEMING,[24] ESQ. (by letter).[24a]—Professor Grinter presents a model arrangement of data and calculations. By making a table of story heights and *K*-values (Table 1) and writing on the diagram of the bent (Fig. 4) at each joint in a rectangle drawn around that joint the percentage factor for adjusting moments, confusion is avoided by the diagram not being cluttered with figures.

To assume that the center girders of a symmetrical bent are hinged at mid-point is an ingenious way of avoiding carry-over moments in these girders.

Professor Grinter's purpose is to present a simplified method of wind analysis. It is not difficult of application, but the designer should understand its limitations. Professor Grinter finds in the lower four stories of the classic Wilson and Maney bent the "criterion ratios" to correspond within a ratio of 11% for adjacent stories. The final or "true" moments are thus found to be within 7% of those obtained by the slope deflection method.

Variations of criterion ratios may be quite different in other cases. M. S. Ketchum, M. Am. Soc. C. E., has offered[25] a problem in which the bending moment of a three-aisle, six-story building are determined by a method outlined by Clyde T. Morris, M. Am. Soc. C. E. Using the *K*-values given and following the method of Professor Grinter, the criterion ratios vary from 5 to 50 per cent. The ratio in a story is sometimes less and sometimes greater than in the story above. Professor Grinter allows a variation of 15 per cent.

For more than twenty years the writer has sponsored, although he did not originate, the two conventional methods of obtaining wind stresses known as the cantilever and the portal methods. At the same time, he has always insisted that super-tall buildings and those of irregular framing should receive special study. Professor Grinter and other writers call attention to "errors" of 30% and even 100%, in the stresses obtained by the conventional methods. What they mean to say is that they differ by these percentages from those obtained by the slope-deflection method. The correctness of the slope-deflection method is vigorously disputed[26] by J. A. Van den Broek, Assoc. M. Am. Soc. C. E. David A. Molitor, M. Am. Soc. C. E., who is often quoted, does not regard the basic assumptions of the method as

[24] Care, American Bridge Co., New York, N. Y.

[24a] Received by the Secretary March 21, 1933.

[25] "Steel Mill Buildings", by M. S. Ketchum, Fifth Edition, 1932, Chapter on "Stresses in Stiff Frames by Moment Distribution".

[26] "Elastic Energy Theory", John Wiley & Sons, Inc., 1931.

"exact."[27] Notwithstanding an occasional assault the consensus of engineering opinion is that it is the most nearly correct of all methods thus far presented. It is not workable, however, for design purposes. The Cross methods, although tedious, are feasible and are valuable for analysis of stresses obtained by other methods. For the purpose of analysis, Professor Grinter's paper is especially valuable.

J. D. GEDO,[28] ESQ. (by letter).[28a]—Under the heading, "The Open-Web or Vierendeel Truss," the author states that fixed-end moments in this type of truss occur only in the horizontal members or chords, "provided the effect of direct stress deformation is to be neglected, as is usual."

To neglect the effect of direct stresses is permissible only if they are small in comparison with the moments. No one would neglect the direct stresses in the case of arches; nor is it permissible to do so in the case of Vierendeel trusses.

The writer solved the example in Fig. 15 by the theorem of least work both by neglecting and considering the effect of direct stresses (in the

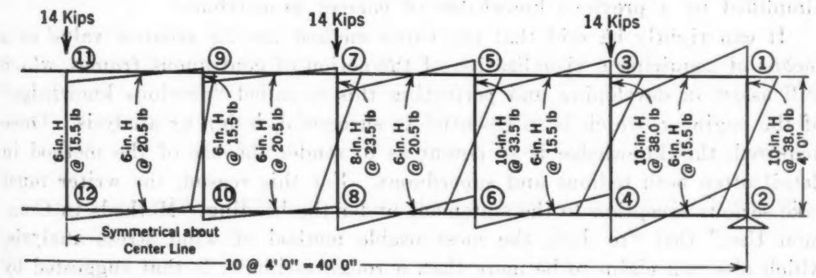

FIG. 15.

chords). In Table 8, which gives the results of this study, the subscripts, *A*, *B*, *L*, and *R*, signify "above," "below," "left," and "right," respectively; for instance, M_{8B} is the moment immediately below Joint 8.

TABLE 8.—COMPARISON OF MOMENTS IN VIERENDEEL TRUSSES

(1 Kip = 1 000 Pounds)

Moment, *M*, at point: (See Fig. 15).	Moment, in Kip-Feet; Direct Stress in Chords:		Moment, *M*, at point: (See Fig. 15)	Moment, in Kip-Feet; Direct Stress in Chords:		Moment, *M*, at point: (See Fig. 15)	Moment, in Kip-Feet; Direct Stress in Chords:	
	Neglected	Considered		Neglected	Considered		Neglected	Considered
1........	35.51	35.10	$5_L = 6_L$....	21.75	20.18	$9_B = 10_A$..	11.26	11.47
2........	35.51	35.10	$5_R = 6_R$....	21.15	22.17	$9_L = 10_L$..	4.37	2.73
$3_B = 4_A$...	55.34	54.73	$7_B = 8_A$....	27.36	27.08	$9_R = 10_R$..	6.89	8.74
$3_L = 4_L$...	20.85	19.83	$7_L = 8_L$....	7.11	5.26	$11_B = 12_A$..	0.00	0.00
$3_R = 4_R$...	34.49	34.90	$7_R = 8_R$....	20.25	21.82	$11_R = 12_R$..	9.63	11.27
$5_B = 6_A$...	42.90	42.35						

[27] *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 189.
[28] Structural Designer, Care, Alexander D. Crosett, Cons. Engr., New York, N. Y.
[28a] Received by the Secretary March 27, 1933.

It is to be noted, in comparing moments in which direct stresses are considered and neglected, that errors as great as 37% (see M_{8L}) can occur.

H. V. SPURR,[29] M. AM. SOC. C. E. (by letter).[29a]—The writer agrees with the author (see "Synopsis") that:

"One of the most difficult problems that the structural engineer must solve is the analysis of stresses in a continuous frame in which joint displacements occur. * * * The calculation of wind moments in tall building frames is such a problem."

He further agrees (see "Use of the Simplified Method of Wind Stress Analysis") that "the experienced designer has a real advantage over the beginner in the use of this method;" or, for that matter, in the use of any method. Furthermore, Professor Grinter is very wise when he states that "the design of any indeterminate structure should be simplified by a previous knowledge of correct proportions." The writer considers it unfortunate that the author did not base his entire paper on these essential facts, since it can be said with equal truth that the anaylsis of an existing structure will be simplified by a previous knowledge of correct proportions.

It can rightly be said that the Cross method has its greatest value as a means of acquiring a visualization of the action of continuous frames, which will assist in developing and perfecting this so-called "previous knowledge" of the engineer, which is so essential to successful design or analysis. Once acquired, this knowledge is so potent as to render the use of the method in detail often both tedious and superfluous. For this reason, the writer must take serious exception to the statement under the heading, "Methods in Common Use," that "to date, the most usable method of wind stress analysis, which also can claim to be more than a rough estimate, is that suggested by Ross and Morris."

It can safely be said that practical design methods are in use in more than one office, that will produce results, the general accuracy of which can not be seriously questioned from the standpoint of analysis. These methods are based on sound mechanics and are used with a visualization of the elastic behavior of continuous frames. Their merit lies in the freedom from arbitrary and incorrect assumptions commonly used in connection with mathematical methods as such. Much confusion and error have occurred in the past from careless assumptions used to make the mathematics apply to a given case.

In the light of "a previous knowledge of correct proportions" how would an experienced engineer initiate his analysis of an existing high building frame? The procedure might be, as follows:

(*a*) Allocate a wind load to each bent that will be tied in later with the design load on the structure.

(*b*) Calculate the horizontal wind shears in the various bents in each story.

[29] Chf. Engr., Purdy & Henderson Co., New York, N. Y.

[29a] Received by the Secretary April 3, 1933.

(*c*) Distribute the horizontal shears into the various panels on the basis of beam distortion only, assuming points of inflection at the mid-span points of the beams. Such shears will cause end moments in the beams, which, in turn, will produce extreme fiber stresses in bending inversely proportioned to the span-depth ratios of the beams (based on their clear spans between columns).

(*d*) From the beam moments the column moments and shears may be obtained, on the basis of points of inflection occurring at mid-story points in the columns.

(*e*) The participation of the various columns in the web distortion may be obtained on the basis of these moments and shears. Their relative values may be obtained, and their relation to beam participation in web distortion as well. If the columns participate equally, or if the column participation is small relative to the beam participation, one may proceed to the next step without review.

(*f*) Determine the relative rigidity of the beams from floor to floor based on the assumed moments. If the floors have approximately the same relative rigidity under their respective moments from wind assumed, then the analysis so far is approximately correct, except for the influence of connections, which is a detail not mentioned by the author. A study of the values of relative rigidities between beams and columns and between floors will reveal how the results are to be modified in specific cases. Frequently, no modification is necessary.

Before any such modification is made, however, it is of vital importance to ascertain the possible influence of change in the length of the columns on the entire analysis. If it is found from the analysis at this stage that the assumed shear distribution will induce vertical stresses in the columns, which, in turn, would cause a vertical movement of the joints in such a manner as to throw the various floors more or less out of line, it is useless to make any further refinement in the analysis without taking such action into account.

How this is to be accomplished the author has not seen fit to establish. In many cases, it will be a weary and tedious task if based on conventional methods of correcting and recorrecting for joint displacement in a high irregular frame. For this reason, the writer has paralleled the procedure outlined by the author with one of his own, up to the point of considering column axial deformation. It is hoped that the points of attack outlined may have some useful significance.

In his discussion[30] of the Second Progress Report of Sub-Committee No. 31, Committee on Steel, of the Structural Division, on Wind Bracing, the writer has pointed out the significance of correct proportions in the design and analysis of multi-story frames, and has indicated a simplified use of the Cross method of analysis under certain circumstances. What needs to be emphasized is that the six steps in the analysis outlined in the present dis-

[30] *Proceedings*, Am. Soc. C. E., August, 1932, p. 1126.

cussion will disclose the type of structure with which one has to deal. The degree of its irregularity will be revealed immediately.

If the structure is regular and well-proportioned the results obtained in the first analysis will be substantially correct, and will be reached in a fraction of the time required by either the general method of successive corrections, or by the author's simplified method. Furthermore, it is important to know as quickly as possible in what manner and do what degree the structure is irregular or unusual. In fact, it is absolutely essential for the engineer to sense these facts, in order to use the simplified method at all, since it is based on: (1) The calculation of the true deflections under the applied loads; (2) producing these deflections by bending in the columns only, with joints fixed against rotation; and (3) balancing the moments in succession at all joints.

It is perfectly true that Steps (2) and (3) are simple of execution, but it is these two steps only which involve the Cross method at all. The real difficulty comes in Step (1) which actually involves the approximate solution of the problem. It is true that under Step (1) the solution obtained may, through lack of insight, be somewhat inaccurate, in which case Steps (2) and (3) will reveal the direction toward which the analysis must move to improve the accuracy. The time consumed in the analysis, however, will depend largely upon the efficiency shown in Step (1), which is all-important.

For this reason the writer believes that progress in the analysis of irregular frames is frequently most rapid through a careful study of deflections or drift produced in the web system by the applied loads, through an intelligent spotting of the positions of the points of contraflexure, and then checking the solution for the condition of minimum deflection in the structure as a whole. This is a common-sense use of the method of least work bearing in mind that the structure will perform under the applied loads in the most rigid manner possible. Frequently, a given structure may have a marked irregularity, but in one respect only, and a reasonable solution is quickly obtained by inspection, or by bracketing the true solution by two analyses somewhat in the same manner that artillery fire is directed.

The author has made a valuable contribution to the art of analysis, but has under-emphasized the degree of art involved in the analysis or design of continuous structures of irregular character. This common error in emphasis is characteristic of nearly all technical papers and discussions on the subject; and it springs from a natural desire of authors and engineers to substitute a definite mathematical procedure for a trained reflection and visualization of the conditions affecting the correct solution.

It should be realized that the actual geometry of deflection is influenced by the clear spans of the members, by the character of the connections, whether angles, split beams, or knee-braces, and, in high towers, that it is largely controlled by the elastic action of the columns under axial stresses. Neither the Cross method nor the slope deflection method are, in themselves only, suitable instrumentalities for many problems encountered in practice.

JOHANNES SKYTTE,[31] ASSOC. M. AM. SOC. C. E. (by letter).[31a]—This paper is of unquestionable value for the analysis of framed structures and particularly for tall buildings acted upon by lateral forces.

In connection with the application of the simplified method for irregular structures, it is of importance to estimate deflections and, for this reason, the following approximate formulas will be introduced: (I) Formula for determining location of point of counterflexure in columns; (II) formula for determining location of point of counterflexure in girders; and (III) formula for determining ratio of girder shears.

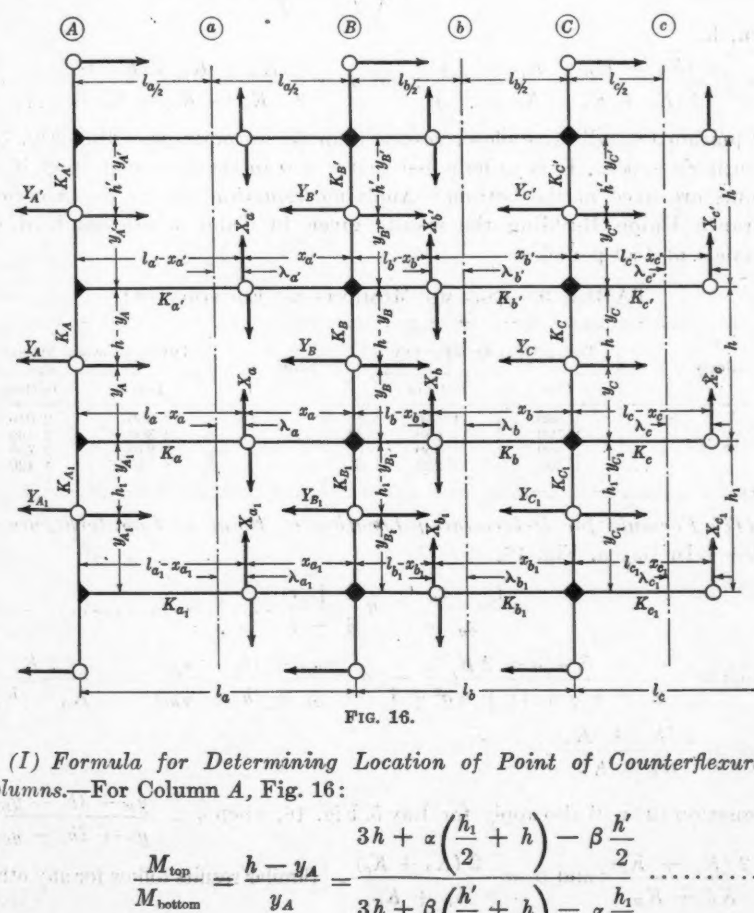

FIG. 16.

(I) Formula for Determining Location of Point of Counterflexure in Columns.—For Column *A*, Fig. 16:

$$\frac{M_{\text{top}}}{M_{\text{bottom}}} = \frac{h - y_A}{y_A} = \frac{3h + \alpha\left(\frac{h_1}{2} + h\right) - \beta\,\frac{h'}{2}}{3h + \beta\left(\frac{h'}{2} + h\right) - \alpha\,\frac{h_1}{2}} \quad \ldots\ldots\ldots\ldots (7)$$

in which, $\alpha = \frac{K_A}{K_a}$ and $\beta = \frac{K_A}{K_{a'}}$. Equation (7) will also apply to Column *B*,

[31] Asst. Hydr. Engr., Hetch Hetchy Project, San Francisco, Calif.

[31a] Received by the Secretary April 24, 1933.

Fig. 16, when $\alpha = \dfrac{K_B}{K_a + K_b}$ and $\beta = \dfrac{K_B}{K_{a'} + K_{b'}}$. Similar results follow for any other interior column.

If a single combined column, restrained by a single girder at each floor level, is considered, then:

$$\frac{M_{\text{top}}}{M_{\text{bottom}}} = \frac{3h + \alpha\left(\dfrac{h_1}{2} + h\right) - \beta\,\dfrac{h'}{2}}{3h + \beta\left(\dfrac{h'}{2} + h\right) - \alpha\,\dfrac{h_1}{2}} \quad \ldots\ldots\ldots\ldots\ldots \text{(8)}$$

in which,

$$\alpha = \frac{(K_A + K_B + K_C + \ldots)}{2\,(K_a + K_b + K_c + \ldots)}\text{; and } \beta = \frac{(K_A + K_B + K_C + \ldots)}{2\,(K_{a'} + K_{b'} + K_{c'} + \ldots)}$$

Equation (8) will give closer results than the formula preceding Fig. 7 of the author's paper. It is to be noted that $\alpha = 0$ in the basement story if the columns are fixed at the bottom. Applying Equation (8) to the American Insurance Union Building the results given in Table 9 are obtained (in thousands of foot-pounds).

TABLE 9.—COLUMN MOMENTS BY EQUATION (8)

Story	Total Column Moment		Story	Total Column Moment	
	Top	Bottom		Top	Bottom
6	1 550	1 550	2	3 030	2 040
5	1 700	1 680	*M*	2 300	2 130
4	1 680	1 650	1	2 810	3 750
3	1 590	2 230	*B*	3 750	4 120

(II) Formula for Determining Location of Point of Counterflexure in Girders.—In Bay *a*, Fig. 16,

$$\frac{l_a - x_a}{x_a} = q\,\frac{1 + r}{3 - r} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(9)}$$

in which, $r = \dfrac{5 + q + 2\,\beta}{2\,(q^2 + q + 1) + \alpha\,q^2 + \beta}$; $q = \dfrac{y_A + (h_1 - y_{A1})}{y_B + (h_1 - y_{B1})}$; $\alpha = \dfrac{2\,K_a}{K_A + K_{A1}}$; and $\beta = \dfrac{2\,(K_a + K_b)}{K_B + K_{B1}}$.

Equation (9) will also apply for Bay *b*, Fig. 16, when $q = \dfrac{y_B + (h_1 - y_{B1})}{y_C + (h_1 - y_{C1})}$; $\alpha = \dfrac{2\,(K_a + K_b)}{K_B + K_{B1}}$; and $\beta = \dfrac{2\,(K_b + K_c)}{K_C + K_{C1}}$. Similar results follow for any other interior bay.

(III) Formula for Determining Ratio of Girder Shears.—Let *b* and *c* be two adjacent bays in a bent; then:

$$\frac{X_b}{X_c} = \frac{K_b\,l_c \pm 6\,\lambda_c}{K_c\,l_b \pm 6\,\lambda_b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(10)}$$

In order to determine the sign of λ, it is necessary to consider oneself located at Column C, Fig. 16; then the plus sign applies when the column and the point of counterflexure in the adjoining girders are on opposite sides of the center line of the same girders. The minus sign applies when they are on the same side of the center line. For the sake of illustration apply this rule to the ratio of girder shears in Fig. 16, then: $\dfrac{X_a}{X_b} = \dfrac{K_a}{K_b}\,\dfrac{1_b - 6\,\lambda_b}{1_a - 6\,\lambda_a}$ and $\dfrac{X_b}{X_c} = \dfrac{1_c + 6\,\lambda_c}{1_b + 6\,\lambda_b}$.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

GEORGE WASHINGTON BRIDGE: CONSTRUCTION OF SUBSTRUCTURE

Discussion

BY EDWARD P. PALMER, M. AM. SOC. C. E.

EDWARD P. PALMER,[5] M. AM. SOC. C. E. (by letter).[5a]—Referring to that part of this paper dealing with the use of continuous belts for transporting concrete materials for the New York anchorage it may be of interest to review some considerations which led to the choice of that method. While the horizontal distance from the unloading dock to the mixers was only 1 000 ft, the ground level at the mixers was 100 ft higher than at the unloading dock, and the top of the storage bins 60 ft above the ground, making the vertical distance which the material was to be transported 160 ft, or about one-sixth as much as the horizontal distance.

The alternate method of transportation considered was the use of trucks for the horizontal distance and the first 100 vertical ft, and a bucket conveyor for the additional 60 vertical ft. It was found possible to lay out a trucking road about 1 300 ft long, with an average gradient of about $7\frac{1}{2}$ per cent. To supply the required material it was estimated that 30 loads per hr would have to be hauled. This required a two-way road, and the 60 crossings per hr over the New York Central Railroad tracks made it appear that it would be necessary to construct a bridge at this point. Owing to the contours, a considerable part of this trucking road, in addition to the bridge, would have to be built on a trestle. Consideration was given to the cost of maintaining such a structure, and, also, in view of the fact that the trucking would be done during the winter months, to the possible difficulties resulting from icy weather.

All these considerations pointed to the choice which was made. The wider sand and gravel belt was placed over the cement belt, giving the cement sufficient protection except during rare, driving rain storms.

NOTE.—The paper by Montgomery B. Case, M. Am. Soc. C. E., was published in January, 1933, *Proceedings*.

[5] Secy. and Treas., Senior & Palmer, Inc., New York, N. Y.

[5a] Received by the Secretary April 6, 1933.

Mr. Case also describes the method of distributing mixed concrete; that is, by means of a belt from the mixers to the base of the tower from which concrete was distributed through chutes in the usual manner. It is here that the writer thinks the plant would have been more efficient had belt conveyors been substituted for chutes. While large chutes were used, and every care was taken to obtain uniformity of mix and a consistency suitable for chuting, the chutes at times were plugged. It appears entirely feasible to design a plant with belt conveyors suspended somewhat in the same manner as the chutes. It is believed that, on account of the low cost of operation, such conveyors would have more than paid for the initial cost through the savings in the height of the tower and in the elimination of lost time necessary to clear the chutes.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

METEOROLOGICAL DATA
PROGRESS REPORT OF SPECIAL COMMITTEE

Discussion

BY C. E. GRUNSKY, PAST-PRESIDENT, AM. SOC. C. E.

C. E. GRUNSKY,[19] PAST-PRESIDENT, AM. SOC. C. E. (by letter).[19a]—The Weather Bureau must not be expected to give out information relating to rainfall or temperature records modified by correction factors. There is no possibility of ascertaining a generally applicable correction factor for any rain-gauge. The best in this particular that can be hoped for is a long-time relation between the catch at two stations, with the chances that this relation will fairly represent that of actual long-time rainfall at these two stations, but not a precise relation during any particular rain storm. It must be remembered in this connection that measurements of rain are at best only approximations and that no two gauges, no matter how near each other, should be expected to catch exactly the same quantity of rain.

The Committee stresses the fact that experience shows that the catch of rain decreases with the altitude of the gauge above the ground. However, there is no fact offered by the Committee which would show convincingly that the record at ground height represents the actual rainfall more closely than a gauge at some altitude. Personal observation compels the writer to believe that spray and drip driven by the wind may add materially to the catch of rain-gauges near the ground, more, of course, than to gauges at considerable elevation above the ground. He is not convinced that, in the various tests which have been made, of which some are cited by the Committee, the catch by the gauge near the ground is the correct standard of comparison.

The writer has twice been connected with the preparation of isohyetose maps of California. For the first map rainfall records at about 220 stations were available. For the second map the station records numbered more than 700. For the first map the records were reduced to represent the average

NOTE.—This Progress Report of the Special Committee on Meteorological Data was presented at the Annual Meeting, New York, N. Y., January 18, 1933, and published in January, 1933, *Proceedings*. Discussion on this report has appeared in *Proceedings*, as follows: April, 1933, by Messrs. C. S. Jarvis, C. F. Marvin, and Ivan E. Houk.

[19] Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

[19a] Received by the Secretary March 28, 1933.

annual rainfall for the 14-year period, 1870 to 1884. For the second map all records were expanded to the 80-year period, 1849 to 1929.

It is remarkable to note that there was scarcely an instance in the preparation of these maps in which the individual record did not fit in with those of neighboring stations. Taken in their entirety the records, whether the gauges were near the ground or on the tops of buildings, deserve the confidence of the engineer. The most glaring discrepancy, curiously enough, which was detected because of non-conformity was a record which was discovered by a prominent engineer and reported by him to the Weather Bureau. When this record would not fit in with its neighbors, a long search disclosed that it was in fact a record taken several hundred miles from the place to which it, in some authoritative publications, still stands accredited.

The Committee's favorable reference (see Part VI) to the record of rainfall kept by Mr. John Pettee, at San Francisco, Calif., is unfortunate, because little dependence is to be placed on this record when compared with the generally accepted record kept by Mr. Thomas Tennent, followed by the records of the U. S. Signal Service and the U. S. Weather Bureau. Knowing this fact, the writer, in August, 1911, obtained the following statement from Mr. Pettee:

"The record was kept wherever I lived; for about twelve years at Francisco and Leavenworth Streets, part of the time in the Mission, part of the time in Hayes Valley, then on Taylor Street between Lombard and Chestnut, and more recently on Clay Street near Baker Street and at present on Cherry Street."

Of these several places, Nos. 1 and 4 are near each other but are about 3 miles from Nos. 2 and 6, and about 2 miles from Nos. 3 and 5. The record is by no means continuous, and the difference in mean annual rainfall at its extreme locations may easily be as much as 1 in.

The approved, down-town, San Francisco rainfall record, on the other hand, is made up of the record by Mr. Tennent from 1849 to 1871, followed by records of the U. S. Signal Service and the U. S. Weather Bureau, continuously located within a few blocks of Mr. Tennent's original location, although, as stated by the Committee, measured in gauges on higher and higher buildings. Mr. Tennent was a maker, or at any rate a repairer, of nautical instruments. He was an experienced and a high-class observer. His record is as consistent and dependable as those which extend it to date. It may be of interest to note that this original record was presented to the writer in 1899 by Mr. Tennent and, after being copied, was turned over by him to Mr. Alexander McAdie, then in charge of the U. S. Weather Bureau Station at San Francisco, for permanent preservation. It is understood that it was lost with other records in the great fire of 1906.

It is to be hoped that the progress report now offered for discussion will bring to the surface other betterment propositions besides those suggested by the Committee. With this in mind, the writer calls attention to several improvements which he believes should be adopted at once.

Whoever has recourse to rainfall records for estimating run-off or run-off rates would welcome a regional rainfall mass curve built up of the daily rain

expressed in percentage of the normal annual rain. Every U. S. Weather Bureau station should issue weekly such a diagram showing the seasonal rainfall to date. The diagram issued at the end of the rain-year would be the one to be preserved. The issuance of such diagrams would involve only a trifle of labor in addition to that which now produces the daily weather sheets, which are of recognized value and which are distributed so generously.

Each such basic mass curve sheet would indicate in words or otherwise, the region to which it is applicable. It would give, in tabular form, the normal annual rainfall for certain places within this region. It would show, by permanent base curve, the mass curve of normal rainfall expressed in percentage of the seasonal normal and applicable to the indicated region. Any point on this curve would show how much of the seasonal rain is expected in a year of normal rainfall, from the beginning of the rainy season to date at that place.

On this base sheet there would be shown from day to day as the season progresses (again expressed in percentage of normal rather than in inches), the daily precipitation at the station of issue in a heavy black vertical line. Thus, for example, if the annual normal is 22 in. at San Francisco, and the mass curve for any year shows that on March 3 at this station 67% of the annual normal has already fallen, and if the record for the following day is 3 in., or 13.7% of the annual normal, then the heavy black vertical line on March 4 would extend from 67 to 80.7 per cent. The public would be served best by having such mass curve sheets covering the record of the previous week issued each Monday.

The final sheet for the rain-year should close of course with June 30, or September 30, or whatever date is adopted for the termination of the local rain-year. This final sheet would be a far more convenient guide to the local and to the regional rainfall than any tabular data. It would have application, as the case may be, to a single station only, or to a considerable area. It could readily be combined with other similar records to construct a dependable regional picture over a large area. The information in percentage of the normal is always readily converted into actual depth of water at all places where the normal precipitation is known.

The writer takes this occasion again to protest against the publication of precipitation by calendar years. In all regions that have successive wet and dry periods the precipitation of any single wet season should not be split at the new year and combined with partial records in other wet periods. By doing this, as is common practice the world over, distorted conclusions are reached. The true range from minimum to maximum, for 12 months, for example, cannot be given. Neither can correct conclusions be reached as to the relation between precipitation and run-off. To illustrate: The actual range of precipitation in 12 months in Central California is from a minimum of about 30% of the annual normal to more than 200% of normal, when the several rain-years, from September 1 or October 1, to the next August 31 or September 30, are compared with each other. By calendar years, the range would appear materially less, because both the extreme dry year and the very

wet seasonal year are split and combined with the partial records of years that are more nearly normal. This results from the fact that practically all the precipitation that is to be taken into account, falls between November 1 and the following April 30. The seasonal or rain-year, as far as the Pacific Slope is concerned, may be considered to begin any time from July 1 to October 1. It is unfortunate that, to the present time, the rainfall statistics for this region have been sent out to the world on a calendar-year basis, thus leading to frequent misuse by the present crop of "cycle" cranks.

In many of his hydraulic problems, the engineer must evaluate as best he can the evaporation loss from open-water surfaces. He is, therefore, in need of basic information relating to evaporation and to methods of determining the evaporation from a water surface not yet in existence, on the basis of known meteorological conditions.

To be helpful in this matter the Weather Bureau has included in its activities observations of evaporation from pans in many sections of the country. However, the records are of little value because the apparatus used is not of the proper type. This statement is made advisedly, based on long experience, and refers to floating pans as well as to the Weather Bureau's standard land pan.

The standard land pan is placed above ground, supported on timbers arranged so as to give air circulation under the pan. Maximum effect of the sun's rays upon the sides of the pan and maixmum air heating or cooling by wind movement are thus secured, to the disadvantage of the desired result. The variable heat from day to day—on partly cloudy days, from hour to hour—which the sun puts into the sides of the pan, even above water, is fed into the water and accelerates evaporation. The record from a pan thus placed does not present a true picture of surface evaporation, nor does it give a record comparable with records in similar pans elsewhere, because sunshine and wind conditions are not the same.

The writer has discussed this subject in connection with the paper by Carl Rohwer, Assoc. M. Am. Soc. C. E.[20] He would urge that the Weather Bureau issue instructions relating to a standard floating pan which, besides being protected against wind-carried spray from the outside, should always have its water surface below the surface of the surrounding water. Furthermore, the floating pan should be in deep water so that its water temperature will correspond as nearly as may be with that of the general water surface. If placed in the shoal warm water inshore, the measured evaporation will always exceed that for the broad open-water surface.

The instructions in this connection might be about as follows: A floating pan for the observation of the rate of evaporation from the body of water in which it floats, should be placed well offshore, so as to be removed from the accelerating effect of warm shoal marginal water. It should be at least 3 by 3 ft square, or preferably 4 ft circular. It should be surrounded by a flat raft of sufficient lateral extent to prevent ordinary strong winds from splashing water into the pan. The pan should have a peg with a pointed top in its

[20] See p. 902.

center to mark the water surface when the pan is full. The depth of water in the pan should be at least 12 in. The sides of the pan should rise 4 to 5 in. above the top of its water surface when full. The pan should be so deeply immersed in the surrounding water that at all times its water surface will be somewhat below the surface of the surrounding water. If there is any possibility of ripples splashing up on the inside of the pan above the level of the outside water, then the sides of the pan should carry a V-shaped rider to keep these sides in the shade. The record should show how much water is required from day to day or from week to week to replenish that lost by evaporation. For this purpose the observer should use a standardized cup both to refill to the top of the peg and to bail out rain water. He should be instructed that extreme precision in bringing the water to the top of the peg is required only when observations are taken immediately before a threatening rain storm and immediately upon cessation of the rain. The errors at other times will correct themselves, when the desired result is a long-time record, as for a month or a year.

It is hoped that these or like suggestions for the betterment of the Weather Service will be approved and submitted by the Committee, and that full credit will be accorded the Weather Bureau for the very valuable service it has already rendered.

The rainfall records should be kept exactly as at present, but, of course, with sufficiently frequent descriptions of the location of the gauge and of the prevailing wind direction during the rainy season so that scientists who desire to make deductions may have the necessary warning to use the records with caution and understanding. At certain population centers, it may be desirable, too, to supplement the station record by measured weekly aggregates of precipitation at a number of especially selected near-by points.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

EVAPORATION FROM WATER SURFACES

A SYMPOSIUM

Discussion

BY MESSRS. RALPH R. RANDELL, C. E. GRUNSKY, AND CHARLES H. LEE.

RALPH R. RANDELL,[24] M. AM. SOC. C. E. (by letter).[24a]—Wind, temperature, and humidity are the only factors the variations of which produce effects on reservoir evaporation sufficiently large to be important in most water supply problems. Therefore, if any collection of true reservoir evaporation data is properly adjusted to a given value of one of these three factors, the adjusted data should plot approximately consistently against the other two factors as co-ordinates.

The writer has investigated considerably, although not exhaustively, the question of whether, and to what extent, the annual evaporation data given in Table 8, of Mr. Follansbee's paper, meet this test. The tabulated annual evaporations were adjusted to fixed values of one or another of the three principal influencing factors on the basis of rational and widely held concepts of the effect of the factor upon evaporation. The annual rates of evaporation, as so adjusted, fail to plot even approximately consistently against the other two influencing factors as co-ordinates.

This failure of the adjusted figures to plot consistently raises a doubt in the writer's mind as to the approximate correctness of the data given in Table 8, and of the pan coefficients on which they are partly based.

Possibly, in the light of their intimate familiarity with the work, the authors, or the members of the Committee or Sub-Committee, may be able to devise an adjustment which will give results that plot consistently. It is earnestly hoped that they will attempt to do so, and will report thereon fully. This seems desirable for two purposes: First, to indicate what degree of accuracy and reliability the data possess; and, second, to put the data in such a form that they can be applied and used most intelligently and conveniently.

NOTE.—This Symposium on Evaporation from Water Surfaces was published in February, 1933, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[24] Senior Engr., Federal Power Comm., Washington, D. C.

[24a] Received by the Secretary March 17, 1933.

It is suggested that, first, all the yearly evaporation rates be adjusted to a common basis—as, for example, to a basis of 60% humidity, or a wind velocity of 4 miles per hr—and that the method of making the adjustment be stated. The adjusted evaporation rates should then be plotted against the two other principal evaporation-influencing factors as co-ordinates, the value of each adjusted rate, in inches per annum, being shown at the plotted point on the graph in small figures. Finally, generalized curves representing yearly evaporation rates of 10 in., 20 in., 30 in., etc., should be added.

Such a graph will show what degree of consistency the data possess, and, together with the adjustment formula on which they are based, will give the best obtainable approximation of the annual rate of reservoir evaporation corresponding to any given average values of wind, temperature, and humidity. Even if the given data plot somewhat erratically, the graph, nevertheless, may be of value in suggesting changes—either in the coefficients, the methods of adjustment, or otherwise—which will bring the data into better or more apparent consistency.

C. E. Grunsky,[25] Past-President, Am. Soc. C. E. (by letter).[25a]—It is with some reluctance that the writer undertakes to contribute his views on the determination of evaporation from large water surfaces to those expressed by the Sub-Committee on Evaporation of the Special Committee on Irrigation Hydraulics in its final report. He feels, nevertheless, that the profession has gone so far astray in its methods of evaporation observation in the past that the data on which the Sub-Committee's conclusions are based, should not be allowed to pass unchallenged, in order that what is dependable information may be segregated from that which is not dependable.

The writer does not agree with some of the conclusions of the Sub-Committee and will give his reasons briefly.

The Sub-Committee states that "if only one [pan] is feasible, the Class A land pan is preferred." The writer contends that this is a type of pan that should never have been brought into use, for the reason that its arrangement is such as to give maximum influence upon evaporation of sunshine and air movement. Even if a more or less consistent relation can be found between the record from such a pan and that from a near-by large water surface, it must be remembered that the heat due to sunshine is fed into the water from all parts of the pan that the sun reaches and that, consequently, the evaporation from an open-water surface, wherever there is much sunshine, must be much less than the evaporation from the pan. This fact is apparent from the data cited, which indicate that generally such a pan will evaporate nearly 50% more water than a near-by open water surface will lose by evaporation.

By exposure to air on all sides and beneath it, maximum effect is given to air temperature changes. There is more warming and more cooling by contact of the air with the sides and bottom of the pan and with the surface of the water in the pan than if the water surface alone were exposed.

[25] Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

[25a] Received by the Secretary April 3, 1933.

The correct type of standard land pan should preferably be circular, about 4 ft in diameter, with water about 12 to 15 in. deep, sunk into the ground to above the water level and provided with an inverted V-shaped rider to shade the sides of the pan down to the ground on the outside and down to the water on the inside. The results of observations from a pan of this type will certainly establish relative rates of evaporation in different parts of the country much better than the Class A pan with its disturbing, constantly varying, sunshine factor.

The Sub-Committee recommends the use of the U. S. Geological Survey floating pan, but fails to state that no floating-pan record should be regarded as dependable and comparable with other records unless the water in the pan is at all times at a lower level than the water outside the pan. When this is not the case, heat is fed into the water of the pan by the sides, which capture heat from sunshine and air, and, in consequence, the pan record must exceed the actual evaporation from the water in which it floats, instead of closely approximating it. The Sub-Committee at least should add to its floating-pan recommendation the condition that the water within the pan at all times must be lower than the outside water. Undoubtedly, even the late Desmond Fitzgerald, Past-President and Hon. M. Am. Soc. C. E., failed to recognize the importance of this requirement, with the result that, at times, he found a difference of as much as a 10° F between the temperature of the water in his tanks and that of the surrounding water.

The experimental observations at the Salton Sea, part of which are reported by Mr. Rohwer in Table 3 (*c*), are to be used with extreme caution. The set of pans, for example, that were termed floating pans in the original record, were not of the correct type. They were purposely so suspended in the water that the water within the pans was 3 to 4 in. higher than the surrounding water. This fact was discovered by the writer some years after the experiments had been made, when he found that observation results would not agree with his own ideas as to evaporation rates in that locality. The only effect of the surrounding water at Salton Sea was to influence in some degree the water temperature in the pans. Unfortunately, these misleading records are being constantly quoted because the true circumstances are not known.

It may be assumed that nearly every floating pan of which there is record was improperly suspended. The endeavor has been to keep the top of the pan so far above the surrounding water that water from the outside will not splash in, and no regard has been had to the fact that when the inside water is above the surrounding water, all the heat fed into the sides of the pan above the water-line goes into evaporation, which, instead of conforming to that of the outside water, is unduly accelerated. Here, too, therefore, a new start is desirable.

The writer calls attention in this connection to the statement by Mr. Rohwer that the Salton Sea pans referred to in Table 3 (*b*) were well immersed. They were not well immersed, and they should not be called floating pans. Referring to observations at Bakersfield ("General Comparisons:

Land Pans and Floating Pans"), Mr. Rohwer decribes the floating pan as having the water on the inside 3 in. above that on the outside. The floating pan at Independence, likewise, is described as having had water at about 2 in. below its rim. The top of the pan was certainly higher than this above the outside water. Here, again, the record has but little value as a floating-pan record. Referring to the records from floating pans taken for the Canadian Pacific Railroad Company, Mr. Rohwer states that the water in the floating pans was kept slightly above the level of submergence. Even at the Fall River Mills, in California (see Table 2(g)), the writer some years ago found the water in the floating pan slightly above the outside water. Consequently here, too, there was undoubtedly slightly more evaporation from the pan than from the surrounding water.

As late as 1930, under expert direction, floating pans were placed in Suisun Bay, California, for a determination of evaporation in connection with studies for a salt-water barrier. The writer called attention to the fact that these were insufficiently immersed and that heat from the sides was being fed into the water of the pan, thus accelerating evaporation. At his suggestion, a comparison with a properly arranged floating pan was made. Although this comparison covered only the two cool months of October and November, nevertheless it showed that the loss from the pan that had its water above the level of the outside water was 10% greater than the loss from a correctly arranged floating pan.

The writer makes the plea that the Sub-Committee revise its report; that the Type A pan be condemned; that the U. S. Weather Bureau be urged to make a new start, in the matter of evaporation observations, with land pans sunk in the ground, with the sides shaded, and with floating pans that have the inside water at all times below the outside water. It is only thus that results may be hoped for, which will stand comparison and which will help to establish some usable relation between meteorological conditions and the total evaporation from large open water surfaces in such periods of time as a month or a year.[26]

Charles H. Lee,[27] M. Am. Soc. C. E. (by letter).[27a]—The Special Committee on Irrigation Hydraulics is to be congratulated upon its accomplishment in producing a study of evaporation which is of practical value to the hydraulic engineer. Its selection of the U. S. Weather Bureau Class A land pan as a standard is thoroughly justified by all the facts. The writer concurs in this selection on the basis of his own experience with various types of pans, especially during the five years, 1928 to 1932. The findings of the Committee regarding the coefficient to be applied to the so-called U. S. Geological Survey floating pan are at variance with the view previously held by many, including the writer, that evaporation observed in such a pan

[26] "Physical Data and Statistics," by William Ham. Hall, M. Am. Soc. C. E., State Engr. of California, 1886, p. 379; *Engineering News*, Vol. 60, August 13, 1908, pp. 163-166; *Bulletin No. 100*, U. S. Dept. of Agriculture, p. 323; *Monthly Weather Review*, 1908 and 1910; and, "Evaporation from Lakes and Reservoirs," by C. E. Grunsky, Past-President, Am. Soc. C. E., *Monthly Weather Review*, Vol. 60, January, 1932.

[27] Cons. Hydr. Engr., San Francisco, Calif.

[27a] Received by the Secretary April 29, 1933.

closely approximates that from the surrounding reservoir or lake surface. The experimental results, however, would seem to support the coefficient of 0.80 adopted by the Committee.

The writer desires to make slight corrections in the descriptions of evaporation pans used by him at Independence, Calif., as stated by the Committee. In Table 5(*f*) the land pan, $3\frac{1}{2}$ ft in diameter and 4 ft deep, is designated as a Type 2 pan. This pan has the same depth and setting as the Colorado land pan (Type 3), and, although circular, has practically the same area. The ratio of evaporation from it to that from the U. S. Geological Survey floating pan (Type 4), in Owens River, is 0.97 (Table 6), which corresponds with the ratio between the Type 3 and Type 4 pans found by the Committee. It is believed that the pan used by the writer should be classed as Type 3.

The land pan, 3 ft square and 10 in. deep, designated as Type 3 in Table 5 (*f*), does not conform to any of the five types considered by the Committee. Although of the same area as the Colorado land pan (Type 3), it was set in a shallow excavation with soil banked up to about one-half the depth of the pan. It corresponded more nearly with the U. S. Weather Bureau land pan (Type 1) than with the Colorado pan, but was smaller and had temperature control from both air and soil. It is believed that the designation, Type 3 land pan, for this pan is incorrect, and that no attempt should be made to classify it.

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DISCUSSIONS

GEORGE WASHINGTON BRIDGE; APPROACHES AND HIGHWAY CONNECTIONS

Discussion

By Messrs. S. Wood McClave, Jr., Wilbur J. Watson, Harold M. Lewis, and Hunley Abbott

S. Wood McClave, Jr.,[2] M. Am. Soc. C. E. (by letter).[2a]—The subject of approaches and highway connections to the George Washington Bridge has been admirably covered in this paper by Mr. Evans. When the New York approach is finished it will be as nearly perfect as can be accomplished by engineers who are confronted with the main difficulties arising from any closely built-up section.

The New Jersey approach, in the Borough of Fort Lee, presented an extremely difficult situation from the municipal standpoint. The original plan of the Port of New York Authority had been approved by the Mayor and Council to the easterly side of Lemoine Avenue and a circular plaza was suggested at Lemoine Avenue because, at that time, no State Highway plans had been made for the traffic expected from the west. The Highway Department had suggested a proposed plan that was not satisfactory to the Borough.

The Governor of New Jersey was consulted, and the Chief Engineer of the Port Authority appointed an Engineering Committee to represent all bodies interested in this approach. After two years of the most difficult kind of work this Committee decided on the present plan, which was immediately accepted by all the governing bodies represented by their engineer.

This approach, with its marginal roads, eliminates the possibility of seeing any backyards from the main highway. It also enhances in value the property immediately facing the approach. It serves the purpose, with right-hand turns only, of taking care of all local traffic, while through traffic can proceed with safety at high speeds without the aid of a traffic light or traffic police. There are no grade crossings to interfere with the traffic, and, consequently, there is no interference with fire protection. This approach should serve as a model for all bridges in the future.

Note.—This paper by J. C. Evans, Esq., was published in February, 1933, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[2] Civ. and Cons. Engr. (McClave & McClave), Cliffside Park, N. J.

[2a] Received by the Secretary March 18, 1933.

WILBUR J. WATSON,[3] M. AM. SOC. C. E. (by letter).[3a]—An important definition of a bridge approach is contained in the paper by Mr. Evans, under the heading, "Approaches as Major Elements of the Bridge Project." It is interesting in this connection to note that the problem of what constitutes a bridge approach actually led to litigation in connection with the Lorain-Carnegie Bridge, in Cleveland, Ohio, built under the auspices of Cuyahoga County.

Before work on the design was begun, the City and County authorities came to a verbal agreement as to the limit to which the County would go in building the approach structure, and it was understood that the City of Cleveland would continue the work from that point. As in the case of the Port of New York Authority and the Highway Department of the State of New Jersey, the broad plan involved the widening and opening of streets several miles from the bridge site, and, consequently, there was no definite limit as to what constituted the approach.

As the work neared completion, it became evident that the City could not abide by its part of the verbal agreement, for financial reasons; but the bridge proper was built for about three-fourths of the estimated cost, mostly because of decreased costs of labor and materials; and, therefore, the County authorities proposed to undertake to complete the approaches as least as far as the available funds would permit.

Legal advisers for the County, however, ruled that money from the authorized bond issue could not be used for this purpose inasmuch as the work contemplated could not properly be included as the bridge approach, the wording of the bond issue being "for the bridge and the necessary approaches thereto." It was finally taken to the Courts in a friendly suit, to determine the meaning of the word "approach."

A diligent search of existing literature indicated that there was no clear and adequate definition of the word other than, for example, "the construction leading to the end of a bridge." Accordingly, the writer offered the following definition to the Court, after certain necessary legal phraseology had been added by the attorneys:

"The approaches to a bridge comprise the traffic arteries leading to the ends of the bridge proper, and such adjustments of alignments and grades of said arteries in the immediate vicinity of such ends as is necessary to afford the maximum convenience of access, and render available to the public the entire capacity of the bridge proper."

As a part of the preliminary studies for the bridge, E. J. McIlraith, Traffic Engineer, of Chicago, Ill., submitted a report in which was contained a remarkably clear statement of the basic principles underlying the design of approaches, as follows:

"Since bridges are necessarily limited in number each becomes of great importance as an artery of travel and the use made of it is dependent on the convenience of the approaches. It is also vital that the streams of traffic

[3] (Wilbur Watson & Associates), Cleveland, Ohio.

[3a] Received by the Secretary March 23, 1933.

flowing to and from a bridge do not cause harmful congestion to business located near the bridge terminal. The arrangements should be such as to spread benefits to all rather than to create damage to some.

* * * *

"The major consideration should be to develop a definite city planning approach which considers the effect of a proposed construction on the increase in usefulness and value of the affected sections, and provides necessary and desirable thoroughfares, routings, and betterments for traffic that will permit and encourage great improvements in freedom of traffic flow. Traffic studies indicate supplementary information to show when each of several probable bridges will become necessary, but the exact location of each bridge and the treatment of its approaches must be based on effective street planning that will serve as part of the ultimate plan for the city's greatest possible growth.

"The general scheme of bridge location and of development of the approaches to the bridge must therefore be set up boldly to fit an outstanding need and in the interest of the greatest civic good."

A comparison of the wording of the report with the definition given by Mr. Evans for the approaches to the George Washington Bridge, shows a remarkable agreement between the two.

HAROLD M. LEWIS,[4] M. AM. SOC. C. E. (by letter).[4a]—In the description of the proposed regional highway system of the Regional Plan of New York and Its Environs, the George Washington Bridge is referred to as the "keystone" of a metropolitan loop highway encircling the central areas of the Region about fourteen miles from New York City Hall. It is on the northerly section of such a loop, which eventually would re-cross the Hudson River at its mouth at The Narrows. This loop has been laid out to connect all the main radial highways in the outer areas, and over sections of it vehicles may gain access to the most direct route to their destination in the central areas. Through traffic from all directions will find it a convenient by-pass around the congested districts. The loop crosses the Arthur Kill by the Goethal's Bridge, another Port Authority structure.

The George Washington Bridge will be used more and more as a by-pass for traffic going through New York City on its way between New England and the Middle Atlantic States, or for traffic between The Bronx, Queens, Brooklyn, or Northern Manhattan, on the one hand, and New Jersey or points west or northwest, thereof, on the other hand. Therefore, the approaches to the bridge are of the utmost importance and must be connected with all radiating trunk highways on both sides of the river.

The State of New Jersey has provided such connections in an admirable way with its system of State highways radiating to the north (Route 1); the northwest (Route 2); the west (Routes 4 and 6); and the south (another section of Route 1). Within ten miles of the George Washington Bridge, the New Jersey State Highway Department is spending approximately $40 000 000. Of course, the highways involved will supply future needs for much traffic that does not cross the bridge, but their construction up to the

[4] Engr., Regional Plan Assoc., Inc., New York, N. Y.
[4a] Received by the Secretary April 6, 1933.

present time (1933) has been promoted primarily because of the presence of the bridge. The State Highway Engineer has estimated that about one-half this total cost is chargeable to bridge traffic.

What is being done on the east, or New York, side? No adequate provision has been made for connecting with trunk-line highways leading to the east or northeast. Of course, the problem was much more difficult on the New York City side of the bridge because adjoining areas were built up and land was many times as expensive. The Port of New York Authority has undertaken to bring a double vehicular tunnel as far east as Amsterdam Avenue in Manhattan, where New York City has projected a serpentine series of connecting roadways within the boundaries of Highbridge Park.

It is just as important to have main highway approaches, free from traffic interruption, connect with the routes to New England, The Bronx, and Long Island, as it is to provide similar connections in New Jersey. In spite of the difficulties from interference with existing developments, adequate future connections, if planned now, should not be impossible. Looking ahead to the completion of the full potential roadway capacity of the George Washington Bridge and the future addition of rail facilities on its lower deck, some provision must be made for extending such facilities to the east by new connections.

A bridge across the Harlem River designed to carry both rails and vehicular roadways provides a logical solution for this problem. The Regional Plan included such a proposed bridge between West 178th and West 179th Streets and a new bridge at this site has been considered by the Department of Plant and Structures of the City of New York and endorsed by local civic organizations.

The original study by the Regional Plan was prepared in 1926. It called for a two-level bridge across the Harlem River with a roadway on the upper deck and provision for future rail facilities on the lower deck. In Manhattan it proposed a depressed roadway in open cut on the north side of West 178th Street, passing under all the avenues east of the George Washington Bridge Plaza. On The Bronx side the bridge roadway connected with University Avenue, the main roadway continuing partly in open cut and partly in tunnel to join East 170th Street at the foot of the bridge between the Harlem River and Jerome Avenue.

Mr. Evans has stated that such a plan for the Manhattan approaches between the George Washington Bridge and Amsterdam Avenue was considered by the Port Authority, but was abandoned due to the amount of property required and to the objection by the City authorities to the destruction of so much property value. The approach plans that were adopted appear to be designed for a traffic movement that will be altered materially when a new Harlem River Bridge is constructed. The efficiency of a temporary arrangement is not as important as that of the final plan.

The Regional Plan proposal for a Harlem River Bridge has been revised to adapt it to the plan for vehicular tunnels under West 178th and West 179th

Streets, as adopted by the Port Authority and the City. It is shown in Fig. 18, on which heavy lines indicate new facilities in addition to those already under construction.

The axis of the proposed Harlem River Bridge would intersect Amsterdam Avenue at the center line of West 178th Street, where a connection would be made with the eastbound tunnel (to be operated temporarily as a two-way tunnel) in that street and the projected future westbound tunnel in West 179th Street. All traffic using these tunnels as a route between The Bronx and the George Washington Bridge would also use the new Harlem River Bridge. Amsterdam Avenue would become an important approach to the George Washington Bridge from the main part of the city to the south.

An exit ramp from the eastbound tunnel is indicated on the north side of West 178th Street, which would permit traffic using it to make a right-hand turn south into Amsterdam Avenue. It is believed that traffic desiring to leave the tunnel at this point could be separated safely from the two lanes of moving vehicles because the small amount of crossing involves only one lane and a movement that is carried out without difficulty or hazard at many points of a similar nature above ground. An entrance from Amsterdam Avenue to the westbound tunnel would be provided by two ramps near 179th Street, one of these for trucks and the other for passenger vehicles avoiding, in this case, any crossing of traffic lanes and providing full visibility for drivers of vehicles on the ramps or in the main tunnel.

As Amsterdam Avenue is about 15 ft lower than Audubon Avenue in this vicinity, these connecting ramps, as shown on the plan (Fig. 18), will readily fit in with maximum grades of 4 per cent. On The Bronx side the highway connections with University Avenue would be approximately level, while the main roadway would pass under University Avenue on a 4% grade. The roadway on the proposed Harlem River Bridge would be at an elevation of 140 ft above mean high water. With provision for future railroad tracks on a lower deck and an arch structure similar to that on Washington Bridge and the rebuilt section of High Bridge, there would be a clearance of about 110 ft over the center of the waterway.

By making Amsterdam Avenue a one-way street, for northbound traffic only, between West 178th and West 181st Streets, as indicated on Fig. 18, there will be no crossing of traffic lanes between these points. Any southbound local traffic on Amsterdam Avenue would be routed through Audubon Avenue for these three blocks.

This plan involves the acquisition of the entire block between West 178th Street, West 179th Street, Amsterdam Avenue, and Audubon Avenue, but it would provide a site sufficient for a future railroad terminal to serve tracks crossing both the George Washington and Harlem River Bridges and future connections between the former and a north and south route in Amsterdam Avenue.

As the route across The Bronx is planned as part of the metropolitan loop highway, previously mentioned, it should have convenient connections with Amsterdam Avenue for vehicles to and from points to the east. This is provided by two ramps in Highbridge Park, one of which would use part

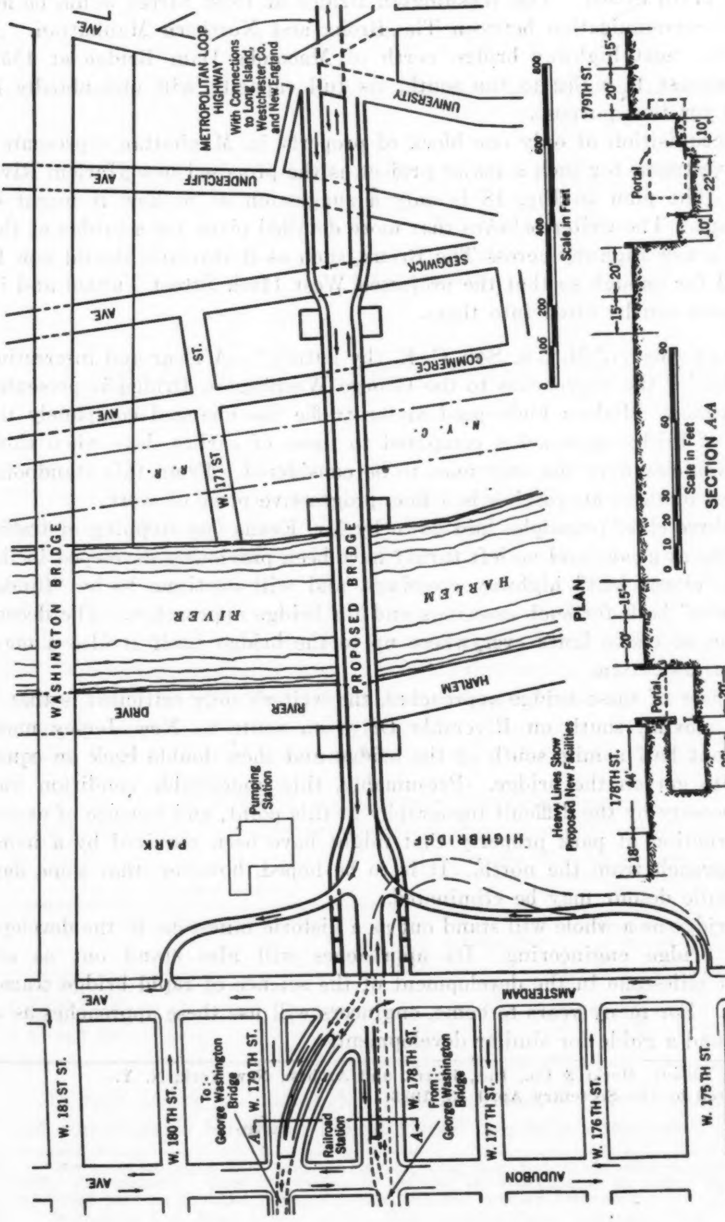

FIG. 18.—STUDY FOR AN EASTERLY EXTENSION OF THE GEORGE WASHINGTON BRIDGE APPROACHES AS PART OF A METROPOLITAN LOOP HIGHWAY.

of that under construction at present (1933) as a connection for the tunnel in West 178th Street. The Washington Bridge at 181st Street would be left for local communication between The Bronx and Northern Manhattan. As this is the next highway bridge north of Macombs Dam Bridge at 155th Street, almost 1½ miles to the south, its full capacity will undoubtedly be required for that purpose.

The acquisition of only one block of property in Manhattan represents a small investment for such a major project as the proposed new Harlem River Bridge. The plan in Fig. 18 is only a suggestion as to how it might be carried out. The writer believes that more detailed plans for a bridge at this site and a new highway across The Bronx, such as it involves, should now be advanced far enough so that the projected West 179th Street Tunnel and its connections can be fitted into them.

Hunley Abbott,[5] M. Am. Soc. C. E. (by letter),[5a]—A clear and interesting description of the approaches to the George Washington Bridge is presented in this paper. Modern high-speed motor traffic has changed completely the problem of bridge approaches compared to those of earlier days when slow-moving vehicles were the only ones to be considered. From this standpoint the design of these approaches is a fine, progressive piece of work.

The three chief principles laid down by Mr. Evans (no stopping of traffic, no crossing at grade, and no left turns) have been previously developed in the so-called "clover leaf" highway crossings, and will continue to be "fundamental laws" both for such crossings and for bridge approaches. The decentralization of traffic lanes everywhere up to the bridge itself is also a most important desiratum.

As a user of these bridge approaches, the writer's only criticism is that a motorist moving south on Riverside Drive en route to New Jersey must travel about half a mile south of the bridge and then double back an equal distance to get on the bridge. Presumably, this undesirable condition was made necessary by the difficult topography at this point, and because of excessive destruction of park property that might have been required by a more direct approach from the north. It is to be hoped, however, that some day this one-mile detour may be eliminated.

The bridge as a whole will stand out as a historic milestone in the development of bridge engineering. Its approaches will also stand out as an important milestone in the development of the science of rapid bridge transportation. For many years to come, engineers will use these approaches as a standard and a guide for similar development.

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[5] Pres., Abbott, Merkt & Co., Inc., Engrs. and Archts., New York, N. Y.

[5a] Received by the Secretary April 13, 1933.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

DISCUSSIONS

DEVELOPMENTS IN REINFORCED BRICK MASONRY

Discussion

BY MESSRS. W. K. HATT, AND C. T. SCHWARZE

W. K. HATT,[24] M. AM. SOC. C. E. (by letter)[24a]—Under the heading, "Characteristics of Brick and Brick Masonry," Mr. Hansen states that the Building Code Committee of the U. S. Department of Commerce has recommended 600 lb per sq in. as the allowable working stress in direct compression on the best brick masonry. The Committee gives a more complete definition of the circumstances in which the high stress of 600 lb per sq in. may be applied; for example, in Section 4, under the heading, "Working Stresses,"[12] the Committee stipulates that "the allowable compressive stresses in brick masonry due to continued live and dead loads" shall not exceed the limits given in Table 3. An exception is made in the case of masonry "laid with smooth level horizontal joints and completely filled vertical joints."

TABLE 3.—ALLOWABLE WORKING STRESSES

Grade of brick; average minimum compressive strength tested flat, in pounds per square inch	ALLOWABLE WORKING STRESSES ON GROSS CROSS-SECTIONAL AREAS, IN POUNDS PER SQUARE INCH:		
	Lime mortar	Cement-lime or natural cement mortar	Portland cement mortar
8 000+	100	300	400
4 500 to 8 000	100	200	250
2 500 to 4 500	75	140	175
1 500 to 2 500	50	100	125

If such joints are thoroughly inspected, and if "the effects of eccentric and concentrated loads and lateral forces" are fully considered, the working

NOTE.—The paper by James H. Hansen, Jun. Am. Soc. C. E., was published in March, 1933, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[24] Prof. of Civ. Eng., School of Civ. Eng., Purdue Univ., Lafayette, Ind.

[24a] Received by the Secretary March 25, 1933.

[12] "Modifications in Recommended Minimum Requirements for Masonry Wall Construction," March 16, 1931.

stresses in Table 3 may be increased by 50 per cent. When masonry is composed of brick of different grades the allowable working stress is required to be that listed for the lowest grade of brick of which the masonry is composed.

C. T. SCHWARZE,[25] M. AM. SOC. C. E. (by letter).[25a]—A future of usefulness for reinforced brick masonry will depend upon normal resumption of construction work. Until business and finance have regained normal activity whatever of science is involved in this revived method of construction should be persistently, although carefully, worked out through research. That there is the possibility of future usefulness for reinforced brick masonry is evident. However, to make such a future possible, a careful study of the behavior of brick masonry in combination with steel and the use of steel to overcome the patent weakness of brick masonry in tension must be made. The profession must know what modification of formulas, proposed in the report of the Joint Committee on Standard Specifications for Reinforced Concrete,[26] must be made to undertake the design of reinforced brick masonry for construction members.

Mr. Hansen has called attention to joint action of steel reinforcement and brick masonry. That the author is justified in recommending the use of construction formulas, or modifications of them, advanced by the Joint Committee on Standard Specifications for Reinforced Concrete, becomes evident after a study of test data. To justify this use, longitudinal joints, where horizontal steel is placed, must be made wide enough to embed the reinforcement completely without endangering the cohesion of the brick masonry.

Professor MacCarthy's tests (Table 1, Items Nos. 21 to 25) indicate that a space of mortar between brick and rod of one-half the rod's diameter would be ample for the development of full bond strength for larger rods. For smaller rods a full diameter on each side would probably be a wise precaution. Stirrups should also be embedded throughout their full length to take, completely, vertical components of diagonal tension through bond.

Of real interest is the comparative tests made by Professor MacCarthy between brick masonry and concrete slabs. It is to be regretted that only one mortar slab and one concrete slab were fabricated and tested. It is suggested as a matter for future research that more comparative tests between reinforced concrete and reinforced brick masonry be made under the same conditions of design and loading. It should be noted that, in the comparative tests, the reinforced brick masonry slab showed considerably more stiffness than the concrete slab. For practically the same ultimate load of between 10 500 lb and 11 000 lb the brick masonry slab deflected less than two-thirds as much as the concrete slab.

Attention is called to this apparently greater stiffness of brick masonry since it verified observations made by the writer during tests of reinforced brick masonry beams for the Department of Buildings, City of New York, under the auspices of Mr. Hansen (Table 1, Items Nos. 44 to 46). The con-

[25] Prof. of Civ. Eng., New York Univ., New York, N. Y.

[25a] Received by the Secretary March 29, 1933.

[26] *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1153.

dition under which these beams were loaded was very unusual. Large platforms had been built to bring the load to the third points of the beams. Loading was effected by building up bricks in interlocking layers by workmen, several of whom were on the platforms placing the bricks. As these men moved about on the platform, the beams rocked back and forth on their bearings. Deflection readings could only be obtained after the men had left the platforms. One of the officials of the Building Department remarked, "What these beams are getting is live load with real impact!" The rocking was much worse for Item No. 44 than for Item No. 45 which may partly account for failure in the former at 2.4 times the design load as compared with 4.8 times the design load in the latter.

During this test the writer was impressed with the behavior of the beams in compression. When failure occurred in Item No. 44 (Table 1) on the compression side, it did so at a point where inferior sand had been used in the mortar, but with a suddenness that was startling. Up to the moment of rupture, the beams exhibited the rigidity in compression noted previously in analyzing Professor MacCarthy's results. This rigidity indicates a factor in favor of reinforced brick masonry over concrete, which is herewith presented as a stimulus to further research and verification.

Many new and astounding facts regarding the behavior of concrete have come to light in recent years. Probably the most serious of these is that of plastic flow. How seriously this may affect the design of compression members, such as columns, is evident from the results of experiments made at Lehigh University and the University of Illinois, as reported by Committee 105, on Reinforced Concrete Column Investigation, of the American Concrete Institute, as follows:[27] "If the load is sustained for some time, the stress distribution changes very rapidly due to plastic yielding and volume of concrete." Approximately from the time when concrete came into general use it became known that a "permanent set" occurred when compression loads were applied. The compression side of a reinforced masonry beam is composed of from 88 to 90%, hard burned argillaceous material throughout its length, which has no plastic flow. Only from 10 to 15% is of cementing material, as compared with 100% in a concrete beam. Apparently, this accounts for the rigidity in compression and low deflection in brick masonry beams. Such beams as Items Nos. 44 and 45 (Table 1) failed by shattering brick along the top courses. Mr. Hansen's words, "gave away suddenly in the manner of a breaking stick," about describes such a failure. More research along this line is needed before definite conclusions may be reached, but it does appear as if the rigidity in compression for such beams was governed by brittleness in the brick when reinforcing steel had been stressed beyond its yield point. This idea is in accord with Mr. Hansen's Conclusions 1 and 2 in a review of Table 1.

The author of this interesting paper makes suggestions as to possible uses for, and advantages of, this type of construction. The beauty and durability of brick are legendary. Earliest uses, from archæological discoveries, were in

[27] Journal, Am. Concrete Inst., Vol. 4, No. 6, February, 1933, p. 277.

the Valley of the Euphrates about 5 000 years ago. Since that time brick masonry has given unbroken and faithful service to Man in nearly all parts of the globe. About a century ago, apparently, the first use of brick masonry, with steel as an aid, came into being. Present-day research has shown almost limitless possibilities for this combination of two time-honored materials of construction. As Mr. Hansen points out, these possibilities are both esthetic and utilitarian, and it is to be hoped that the present stimulus in research will demonstrate both feasibility and economy in construction work for reinforced brick masonry.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

THE PLAN OF BOSTON, MASSACHUSETTS: A CAPITAL CITY

Discussion

BY ROBERT WHITTEN, ESQ.

ROBERT WHITTEN,[9] ESQ. (by letter).[9a]—In this excellent paper the author has referred briefly to the Thoroughfare Plan approved in 1930 by the Boston Planning Board. An outline of this plan is shown as Fig. 3 in Mr. Comey's paper. It is based primarily on a recognition of the need for a modernization of the present highway system by the development of a limited mileage of express roads and parkways of generous width, permitting a continuous flow of traffic. The widening of existing streets and the creation of traffic streets of the ordinary type have not been neglected, but these improvements, unless supplemented by those of the express-road type, cannot prevent a further increase in congestion, to say nothing of providing an effective relief for existing congestion.

Just as, about thirty-five years ago, street-surface congestion in Central Boston forced a start in the building of rapid subways to supplement the street surface car lines, so in this day the congestion produced by the automobile is certain to force the supplementing of the ordinary street system by a system of express roads.

A central feature of the Thoroughfare Plan is a great north-south road extending from the northerly city line bordering Revere, to the southerly city line at Readville, a distance of 13.7 miles. It will connect the State Highway System serving Revere, Lynn, Salem, Beverly, and other North Shore cities and towns with the State Highway System serving Stoughton, Taunton, Fall River, New Bedford, and neighboring towns on the south. It will also connect with a proposed new highway to Providence, R. I.

An important feature of the proposed Central Artery and its southerly extension by way of the Blue Hills Radial, will be an upper-level roadway or

NOTE.—The paper by Arthur C. Comey, M. Am. Soc. C. E., was presented at the meeting of the City Planning Division, Boston, Mass., October 10, 1929, and published in March, 1933, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[9] Planning Consultant, and Consultant, Boston (Mass.) Planning Board, New York, N. Y.

[9a] Received by the Secretary April 26, 1933.

viaduct extending from the North Station at Nashua Street, Boston, to the Dover Street Bridge, a distance of about 2 miles. This six-lane viaduct will have a capacity of 60 000 vehicles per day, traveling at an average speed of 30 miles per hour. It will by-pass the chief centers of congestion and will attract to itself approximately 40% of the vehicles that are now (1933) clogging the surface streets of the central area.

The principle underlying the design of main trunk highways and parkways in the proposed Thoroughfare Plan is that they should provide, in so far as it is economically feasible, for a free and continuous movement of traffic. This requires that roadway levels be separated at the more important intersections. On the proposed Central Artery there will be a central viaduct over all cross-streets. On the proposed parkways, provision is made for over-passes or under-passes at all important intersections. For the Blue Hills Radial, the Roxbury Crosstown, and other major routes, a type of broad express road is recommended. For the locations selected this type of express road is believed to be the most economical and efficient means of providing the required traffic capacity, due consideration being given to the importance of speed, safety, and comfort of travel.

The problem presented is both that of providing relief for existing congestion and of providing traffic capacity for approximately 60% more vehicles than now use the city's streets. One way to meet this problem would be to widen a large number of existing streets from their present widths of 50 or 60 ft to 80 or 100 ft. Such wholesale widenings would be enormously expensive and would not remove the chief source of congestion and delay which results from crossings at grade. Under the conditions disclosed as the result of a careful anaylsis of the Boston traffic problem the construction of a few broad express roads seemed to be the only logical and practical solution.

A street system should not be designed to promote either centralization or decentralization. It should be designed to promote safety, comfort, and speed of movement, between all parts of the community. This will result in the best and most economical and orderly organization of the social and business life of the community. It will, in general, promote centralization where centralization is justified and decentralization where that is consistent with the best organization for the community as a whole.

A comprehensive thoroughfare plan is just as essential to the prosperity of the local business centers as it is to that of the main center. It is just as important to the local center that there should be adequate traffic-ways between it and the main center and between it and the other sub-centers as it is to the main center that it should be connected with all parts of the community.

It is sometimes said that it is useless to increase street capacities in central areas, as any additional capacity provided will be taxed immediately to the saturation point. This assumption may be valid in certain situations as applied to local business streets, but has no validity whatever as applied to any major traffic artery—to any street that is an essential part of a compre-

hensive thoroughfare plan. A serious slowing down of the traffic movement in any part of the main arterial system affects the entire community injuriously.

The automobile cannot serve the function now performed by rapid transit subways and elevated roads. These facilities must be relied upon for mass transportation along the most concentrated routes of passenger travel. Together with the surface cars and buses, they must continue to be the main reliance for the daily workward and homeward travel to and from Boston proper. The rapid transit system should be extended so as to reduce the rush-hour street traffic. There should be rapid transit routes and facilities to draw all trips that normally can be made more quickly, conveniently, and economically, in that way; and there should be express roads and parkways to accommodate all trips that normally can be made best by the use of the automobile.

The street system of the city should be adapted to the requirements of a motor age. The art of street design and construction has lagged far behind the art of vehicle design and construction. As a result, the citizens and business men of Metropolitan Boston are being denied the full advantage of one of the most marvelous developments of the age, the motor vehicle.

Industrial and commercial growth throughout Metropolitan Boston is dependent, among other things, on the development of a complete system of adequate and convenient traffic routes. The greatest potential advantage of a factory location in any great metropolitan center is due to the large home market within easy trucking distance. This is one of the chief factors in the continued growth of these great centers. Business growth and prosperity, in turn, are dependent on industrial growth. Convenient access between all parts of the Boston Metropolitan Area by truck and automobile is a prime essential for business and industrial prosperity.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

HIGH DAMS ON PERVIOUS GLACIAL DRIFT

Discussion

BY MESSRS. CHARLES W. SHERMAN, AND F. B. MARSH

CHARLES W. SHERMAN,[5] M. AM. SOC. C. E. (by letter).[5a]—The statement under "Design and Settlement of Embankments" that, for an embankment of pervious sand, the plane of saturation generally "remains below a 1 on 5 slope from head-water to tail-water," provides a convenient approximation, but does not go far enough to be of particular significance. Referring to this slope as a "percolation factor," Mr. Burd states that "finer materials require higher factors," that is, flatter slopes of the surface of saturation. Obviously, this is incorrect unless drainage is not provided in the down-stream part of the embankment; and obviously, too, the slope is a function of the quantity of water percolating through the embankment as well as the porosity of the earth of which it is built, not to mention the tightness of the cut-off wall. If an absolutely water-tight membrane were provided at the up-stream face, no water could penetrate the embankment; the percolation factor (as used by the author) would be zero at the membrane and infinity for the remainder of the distance to tail-water.

Four typical cases from the writer's experience show the following inclinations of the surface of saturation: 1 on 7.5 for a small dam built of coarse sand on a foundation of the same material, with an incomplete cut-off wall; 1 on 5 for a dam built of medium sand upon the same material, with a reasonably good cut-off; 1 on 2.6 for a dam of comparatively dense hardpan, and having a core wall about which little is known; 1 on 1.7 for a dam of dense hardpan, well compacted, but having no core wall, and provided with ample drainage at the toe of the slope. The quantities of water percolating were appreciable in the first case; noticeable but of no special significance in the second; too small to measure in the third (although the undrained toe was saturated); and impossible of detection in the fourth case. The line of saturation in the last example was determined by well pipes driven into the dam.

NOTE.—The paper by Edward M. Burd, M. Am. Soc. C. E., was published in April, 1933, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[5] Cons. Engr. (Metcalf & Eddy), Boston, Mass.

[5a] Received by the Secretary April 19, 1933.

These comments do not in any way reflect upon the interest and value of the paper in so far as it relates to the practicability of building a high and reasonably tight dam upon pervious foundations.

F. B. MARSH,[6] M. AM. SOC. C. E. (by letter).[6a]—In experienced hands there is no reason why structures like those described should not be both feasible and safe. However, they do call for more thorough exploration and understanding of the underground conditions than would be the case if the cut-off were being carried down to rock. Safety depends primarily on the adequacy of the drainage system and its uncloggable effectiveness under all parts of the down-stream toe, whatever the variations in foundation conditions.

In the last few pages of a paper[7] describing the Scituate Dam of the Providence, R. I., water system, the writer suggested the virtues of locating the cut-off at the up-stream toe. Time and money can be saved by using steel sheeting for the cut-off, rather than trying to make it completely watertight, as long as ample provision is made in the down-stream toe for draining all seepage water. Having been nurtured in a conservative eastern environment and being already classed by some as a heretic, the writer had not dared suggest that there might be locations where such a design was feasible even if the rock was too deep to be reached by the cut-off. Now, it has actually been demonstrated. That is progress.

There are many shades of conservatism. For example, there are those who still hesitate to rely on a line of steel sheeting for a cut-off under an important dam, even where the sheeting can be driven to rock. For the consideration of such persons the writer would like to suggest a double line of steel sheeting, say, 20 ft apart, starting either from the original surface, or from the bottom of an open trench of any convenient depth. By providing a sump between the two lines of sheeting, it would then be possible to lower the ground-water in this area and, by measuring the pumpage, obtain an approximate idea of the tightness of the sheeting. Grout pipes could then be put down outside both lines of sheeting, and continued pumping would tend to draw the grout through the points where the sheeting was not tight to the rock. The amount of pumpage should give a fairly direct measure of the effectiveness of the grouting, which could be repeated as often as necessary. In a long structure, such a cut-off could be left with provision for several cross-lines of sheeting, which could be driven later to divide it into bays to be tested separately as desired.

There are many locations where deep cut-offs of steel sheeting are impracticable, because of the presence of boulders or large beds of coarse gravel. Where conditions are favorable to driving steel sheeting it certainly offers economies in cost of construction. If any considerable depth is to be penetrated, the steel section will have to be heavy to withstand the driving operations. With a substantial steel section there seems no question as to the permanence of the cut-off. Even if ultimately, the steel should rust through entirely, the rust would occupy a greater space than the original steel and would still be effective as a cut-off.

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[6] With Board of Water Supply, New York, N. Y.

[6a] Received by the Secretary April 26, 1933.

[7] *Journal*, New England Water Works Assoc., Vol. XL (1926), p. 477.

[illegible]

[illegible] a long structure, such a cut-off could be left with provision for several lines of sheeting, which could be driven later to depth, if cuts have to be [illegible] separately as desired.

There are many locations where deep cut-offs of wood sheeting are impracticable, because of the presence of boulders or large beds of coarse gravel. Where conditions are favorable to driving wood sheeting it certainly offers advantages in cost of construction. If any considerable depth is to be penetrated, the steel section will have to be heavy to withstand the driving operation. With a substantial steel section there seems no question as to the permanence of the cut-off. Even if ultimately, the steel should rust through entirely, the rust would occupy a greater space than the original steel and would still be effective as a cut-off.

With Board of Water Supply, New York, N.Y.

Received by the Secretary April [illegible], 1932.

Journal, New England Water Works Assn., Vol. XL [illegible], p. 477.

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer,* and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibility. It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from May 15, 1933.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct important work	35 years	12 years*	5 years of important work
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years of important work
Fellow	Contributor to the permanent funds of the Society			

* Graduation from a school of engineering of recognized reputation is equivalent to 4 years of active practice.

† Membership ceases at age of 33 unless transferred to higher grade.

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

FOR ADMISSION

ALDRICH, EUGENE VOLNEY, Dos Cabezos, Ariz. (Age 26.) Jun. Highway Engr., U. S. Bureau of Public Roads. Refers to F. W. Flittner, F. C. Kelton, G. L. McLane, C. H. Sweetser.

ANGELL, LESTER WILLIAM, Three Mile Bay, N. Y. (Age 26.) Refers to C. C. Cassel, W. J. Farrisee, R. F. Hall, H. W. Preston, F. C. Wilson.

BHOLE, SHANKER JAIRAM, Dover, N. J. (Age 36.) Refers to F. Auryansen, C. W. Coote, E. G. Hooper, S. Johannesson, C. T. Schwarze, J. F. Williamson.

BLISS, JOHN HARVEY, Santa Fe, N. Mex. (Age 29.) Engr., State Engr.'s Office. Refers to E. L. Barrows, F. P. Goeder, B. Johnson, G. M. Neel, R. L. Parshall, C. Rohwer, W. H. W. Yeo.

BRYAN, LESTER LEON, Washington, D. C. (Age 37.) Hydr. Engr., U. S. Geological Survey. Refers to G. C. Baldwin, L. Crandall, N. C. Grover, W. G. Hoyt, B. E. Jones, C. A. McClelland, T. R. Newell, C. G. Paulsen, H. Stabler.

CALDWELL, LESLIE CHARLES, New York City. (Age 29.) Refers to J. L. Bogart, G. W. Fuller, W. H. Halsey, R. H. Randall, E. R. Smith, H. S. Van Scoyoc.

CUTCHIN, BRAXTON MURRAY, Jr., Fredericksburg, Va. (Age 22.) Timekeeper, Virginia Dept. of Highways. Refers to J. A. Anderson, W. Marshall.

ELLIS, ALBERT RALPH, Pittsburgh, Pa. (Age 52). Director and Vice-Pres., Pittsburgh Testing Laboratory. Refers to J. W. Cowper, C. S. Davis, G. C. Diehl, S. Eckels, P. J. Freeman, F. P. McKibben, H. N. Ogden, C. M. Reppert, R. M. Riegel, H. E. Riexinger, N. S. Sprague, S. J. Stone, E. B. Whitman.

ETTINGER, LOUIS JOSEPH, Jr., Skaneateles, N. Y. (Age 26.) With Onondaga County Highway Dept. Refers to G. A. Helmstetter, L. Mitchell, S. D. Sarason.

GONZALEZ MANUEL FRANCIS, Pensacola, Fla. (Age 24.) Refers to R. L. Bannerman, C. Phillips, J. E. Soule, L. E. Thornton, W. E. Wheat.

GUTHRIE, MARION CULBERTSON, Walton, Kans. (Age 26.) Refers to G. W. Bradshaw, J. O. Jones, W. C. McNown, H. A. Rice, F. A. Russell.

HANSON, THOMAS COOPER, Detroit, Mich. (Age 27.) Instructor, Dept. of Civ. Eng., Univ. of Detroit. Refers to P. G. Brown, J. H. Cissel, E. L. Eriksen, G. P. Springer, R. B. Wiley.

HSU, SHIH-TA, Tientsin, China. (Age 37.) Executive Member and Chf. Engr., North China River Comm. Member, Hai Ho Improvement Comm., Lecturer, Pei Yang Univ., etc. Refers to N. D. Baker, T. Y. Chen, C. P. Hsueh, J. H. Lance, S. T. Li, H. H. Ling, H. N. Ogden, A. Potter, P. L. Lang.

LIGGETT, ALBERT PARKER, Chicago, Ill. (Age 23.) Refers to E. Boyce, G. W. Bradshaw, J. O. Jones, W. C. McNown, H. A. Rice, F. A. Russell.

McLEOD, DONALD EDWARD, New York City. (Age 24.) Refers to W. S. Lohr, L. Perry, E. H. Rockwell.

MILLS, BRUCE HOPF, Fillmore, Cal. (Age 37.) Civ. Engr., Rancho Sespe. Refers to H. W. Dennis, V. M. Freeman, G. B. Heywood, A. B. Pierce, F. Thomas.

PERSONS, DAVID WAITE, Vicksburg, Miss. (Age 26.) With Mississippi River Comm. Refers to F. G. Christian, G. R. Clemens, H. W. King, G. H. Matthes, C. O. Wisler, N. E. Wolfard.

SMITH, W SHERMAN, Toledo, Ohio. (Age 41.) Asst. Prof. of Civ. Eng., Univ. of Toledo. Refers to G. Champe, W. S. Dix, S. C. McKee, L. T. Owen, R. H. Randall, C. L. Sawyer, W. J. Sherman.

STOREY, ARTHUR LIPSCOMB, Houston, Tex. (Age 23.) Chainman, United Gas Public Service Co. Refers to J. S. Broyles, L. E. Grinter, J. T. L. McNew, J. B. Spangler.

TSATSOS, ALEXANDER GEORGE, Serres, Greece. (Age 27.) Asst. Supt. with John Monks & Sons—Ulen & Co., New York and Lebanon, Ind., Contrs. of Greek Govt. Refers to A. B. Christensen, R. W. Gausmann, R. H. Keays, T. S. Shepperd, P. D. Troxler.

WHEELER, JOHN WARD, Indianapolis, Ind. (Age 39.) Member, Indiana State Highway Comm. Refers to W. P. Christie, E. A. Clark, E. F. Collins, W. P. Cottingham, W. K. Hatt, W. A. Knapp, R. B. Wiley.

WHITE, GEORGE CLIFFORD, Pasadena, Cal. (Age 23.) Refers to J. C. L. Fish, E. L. Grant, C. Moser, L. B. Reynolds, J. B. Wells.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

BARNES, GEORGE ERIC, Assoc. M., Flushing, N. Y. (Elected Junior Oct. 15, 1923; Assoc. M. March 5, 1928.) (Age 35.) Asst. Engr. of Design, Div. of Eng., Dept. of Sanitation, New York City. Refers to C. C. Covert, W. Donaldson, H. P. Eddy, E. J. Fort, G. W. Fuller, R. H. Gould, J. H. Kimball, H. D. Mendenhall, M. Pirnie, P. L. Reed, G. T. Seabury, C. M. Spofford.

BRALOFF, HERMAN MATTHEW, Assoc. M., Brooklyn, N. Y. (Elected July 12, 1926.) (Age 35.) Vice-Pres. and part owner, Brader Constr. Corporation. New York City. Refers to J. Feld, I. L. Gelder, C. Goodman, G. Paaswell, R. Ridgway, D. C. Serber, L. White.

BROWN, CHANNING BOLTON, Assoc. M., Charlotte, N. C. (Elected Junior May 28, 1923; Assoc. M. Dec. 15, 1924.) (Age 35.) Designer with Duke Power Co. and W. S. Lee, Cons. Engr. Refers to F. H. Cothran, B. S. Drane, A. C. Lee, W. S. Lee, J. N. Pease, R. Pfaehler.

O'REILLY, ANTHONY RAUEN, Assoc. M., Reading, Pa. (Elected Junior Dec. 6, 1920; Assoc. M. Nov. 14, 1927.) (Age 35.) Chf. Engr., Bureau of Water. Refers to G. S.

Beal, R. C. Dennett, I. M. Glace, H. E. Moses, A. L. Reeder, W. L. Stevenson, G. F. Wieghardt.

PANHORST, FREDERICK WILLIAM, Assoc. M. Sacramento, Cal. (Elected Jan. 15, 1923.) (Age 40.) Acting Bridge Engr., Bridge Dept., California Highway Comm. Refers to C. E. Andrew, R. E. Davis, M. L. Enger, C. S. Pope, C. H. Purcell, F. T. Sheets, A. N. Talbot.

SCHIEBER, OLIVER JAY, Assoc. M., San Marino, Cal. (Elected April 25, 1921.) (Age 44.) Sr. Engr., Metropolitan Water Dist. of Southern California, Los Angeles, Cal. Refers to A. F. Blight, H. W. Dennis, J. Hinds, W. E. Jessup, D. H. Redinger, G. C. Ward, W. Ward.

WILSON, WILLIAM MUNSEY, Assoc. M., College Station, Tex. (Elected Aug. 27, 1928.) (Age 38.) Chf. Structural Engr. for F. E. Giesecke, Coll. Archt. for Texas Agricultural and Mechanical Coll. Refers to R. J. Cummins, P. M. Geren, F. E. Giesecks, F. C. Heins, C. E. Sandstedt, T. U. Taylor.

FROM THE GRADE OF JUNIOR

BETTERLY, JOHN AUSTIN, Jun., Scranton, Pa. (Elected Nov., 12, 1928.) (Age 27.) With Dravo Contr. Co., Neville Island, Pittsburgh, Pa. Refers to C. L. Appleton, C. B. Jansen, C. B. Ruckdeschel, J. S. Shute, C. H. Stevens.

CARLSON, ROY WASHINGTON, Jun., Berkeley, Cal. (Elected Nov. 12, 1928.) (Age 32.) Research Engr., Univ. of California. Refers to W. L. Chadwick, R. E. Davis, E. C. Eaton, D. C. Henny, L. C. Hill, F. A. Noetzli, B W. Steele.

HANNIGAN, EDWARD JOHN, Jun., Lillebonne, S. I., France. (Elected March 28, 1932.) (Age 28.) Field Engr., J. G. White Constr. Co. Refers to C. L. Appelton, T. A. Currie, Jr., W. B. Evans, C. B. Ruckdeschel, J. R. Sharp.

McCULLOUGH, JAY WHEELOCK, Jun., Denver, Colo. (Elected Aug. 28, 1922.) (Age 32.) Civ. Engr. Refers to H. S. Crocker, C. L. Eckel, W. B. Freeman, N. W. Funk, S. O. Harper, M. C. Hinderlider, E. W. Raeder, O. T. Reedy, C. Sacra, R. G. Shankland, W. R. Weber.

MITRA, SURENDRA NATH, Jun., Calcutta, India. (Elected Dec. 14, 1925.) (Age 32.) Designer, Dorman Long & Co., Ltd. Refers to O. W. Crowley, W. E. Jones, Jr., R. K. Kittredge, B. J. Lambert, F. A. Nagler, F. R. White, S. M. Woodward.

MUGLER, RICHARD CARL WILLIAM, Jun., Riverdale, N. Y. (Elected Oct. 21, 1924.) (Age 33.) Pres. and Mgr., Richard C. Mugler & Co., Inc., and Treas. and Mgr., Heydt-Mugler Co., Inc., Bronx, N. Y. Refers to W. J. Barney, G. G. Blackmore, J. Feld, A. T. Glassett, L. F. Hewett.

RUMBLE, GEORGE BERTYL, Jun., Philadelphia, Pa. (Elected July 12, 1926.) (Age 30.) Asst. Engr. with Albright & Friel, Inc. Refers to H. M. Freeburn, F. de S. Friel, I. M. Glace, J. B. Hoke, J. D. Justin, C. F. Mebus.

TSENG, TSAO WEN, Jun., Soochow, Ku, China. (Elected June 9, 1930.) (Age 32.) Sec. Engr., Hua-Ying-Hua-Chow Sec., Lung Hai Ry. Refers to E. E. King, H. H. Ling. (Applies in accordance with Sec. 1, Art. 1, of the By-Laws).

WAGENER, FREDERICK WILLIAM, Jun., Columbia, S. C. (Elected June 26, 1931.) (Age 29.) Asst. Hydr. Engr., U. S. Geological Survey. Refers to E. D. Burchard, N. C. Grover, A. E. Johnson, W. R. King, C. G. Paulsen, F. C. Snow.

The Board of Direction will consider the applications in this list not less than thirty days after the date of issue.